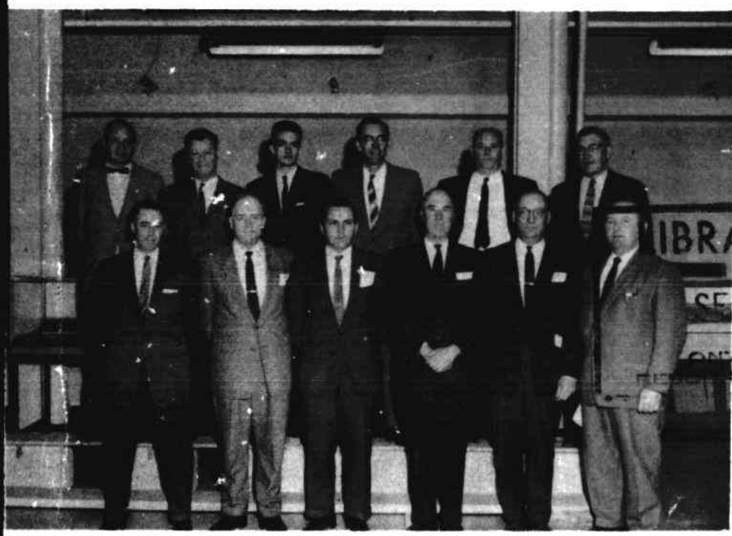
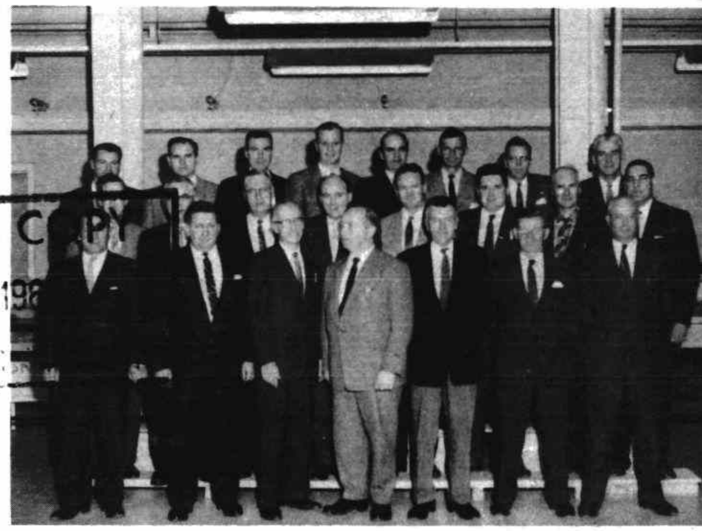



INTERMEDIATE





SEWAGE WORKS COURSE


NOTES

of the

INTERMEDIATE

SEWAGE WORKS OPERATORS' COURSE

MARCH 5th to 9th, 1962

ONTARIO WATER RESOURCES COMMISSION

INTERMEDIATE
SEWAGE WORKS OPERATORS' COURSE

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SEWAGE WORKS OPERATORS' COURSE

ONTARIO WATER RESOURCES COMMISSION

Intermediate Course
March 5 - 9, 1962

Introductory Statement by Dr. A. E. Berry, General Manager OWRC

When the courses for sewage works operators were initiated, it was indicated that these would be organized into three groups, the first one being a basic course, the next an intermediate course, and the final one a senior course leading to a certificate of qualification. This program has been advanced another degree and this intermediate course is offered to those who have taken the first of them.

The objective in this course is to advance the student further along the course originally planned. Different subjects are provided and additional aspects of the field are discussed in the lectures arranged for the Water Resources Commission Building. The subjects will also be more advanced, and the operator will be asked to listen closely to the lectures given and to study these for future training in his field of operation. Once again the operators taking the course will be asked to take an examination at the conclusion of the course. It is essential that the operator be well qualified and that when he completes the final course, and is issued a certificate of qualification, he will have merited this through attendance at the lectures and through the examinations he has written.

This intermediate course will be similar in general to the procedure followed in the initial one, but of course the subjects and the contents will vary. Following the intermediate course arrangements will be made after a period, to advance the operator to the final stage where he can qualify for certification.

The facilities of the Ontario Water Resources Commission are made available for the training of sewage plant operators. These include not only the building and the laboratory but the staff of the Commission. The importance of plant operation needs to be emphasized continuously. The desired results cannot be obtained merely by constructing a plant and paying little, or ineffective attention to its operation. If the works on which money, in considerable quantity, is being spent by the municipalities of this province are to be effective in controlling stream pollution these plants must be operated to the maximum efficiency. The Commission expects that the operators will meet this challenge. These courses are a contribution of the Commission towards this objective of clean streams for the province. We wish you success in the interesting work which you have chosen. You have an important obligation to the public.

X

MATHEMATICAL PROBLEMS IN SEWAGE

TREATMENT & DISPOSAL

by

L. SOUTH

District Engineer

An Address To
The Ontario Water Resources Commission
Intermediate Sewage Works Operators' Cpurse
Toronto, Ontario
March 5, 1962

MATHEMATICAL PROBLEMS IN SEWAGE TREATMENT & DISPOSAL

by

L. SOUTH

District Engineer

INTRODUCTION

In order to carry out routine tests on sewage, assess present operating conditions and decide on possible future treatment alternatives it is necessary to make mathematical calculations. The following information is for reviewing basic mathematics and introducing standard problems and solutions which you can expect to encounter in your routine work.

FRACTIONS

In solving fractions we have to use algebra, which is merely the expression of the relations and properties of numbers by means of letters, signs and symbols. This is generally expressed in the form of an equation. The important point to remember about equations is that once they are established you can only alter them by doing exactly the same thing with both sides of the equation, whether it is multiplying, dividing, adding or subtracting.

2/6 is a fraction in which 2 is the numerator and 6 the denominator.

In order to solve this fraction we must put it in equilibrium or into an equation.

The fraction 2/6 is equal to some number as yet unknown. We will let the letter A represent this unknown, we could use B or any other letter or symbol. In any case we now have an equation which we can solve.

Example 1

$$2/6 = A$$

$$A = 6\overline{)2.000} = .333$$

Example 2

$$\frac{\frac{3}{4}}{\frac{1}{2}} = A$$

$$\text{or } A = \frac{(4\overline{)3})}{(2\overline{)1})} = \frac{.75}{.5}$$

$$\text{or } A = 5.\overline{)7.5} = 1.5$$

The above can be simplified by knowing that the same results can be achieved by inverting the fraction in the denominator and multiplying it by the fraction in the numerator, thus

$$A = \tfrac{3}{4} \times 2/1 = 6/4 = 1.5$$

Example 3

$$\tfrac{3}{4} + \tfrac{1}{2} = A = \tfrac{3}{4} + 2/4 = 5/4 = 1\tfrac{1}{4}$$

$$\text{or } \tfrac{3}{4} - \tfrac{1}{2} = A = \tfrac{3}{4} - 2/4 = \tfrac{1}{4}$$

that is you can only add or subtract fractions with a like denominator.

Example 4

Multiply numerators by numerators and denominators by denominators, thus in the equation

$$2 \times \tfrac{3}{4} = 12 \times 1/8$$

or $$2/1 \times \tfrac{3}{4} = 12/1 \times 1/8$$

solving $$\frac{2 \times 3}{1 \times 4} = \frac{12 \times 1}{1 \times 8}$$

$$\frac{6}{4} = \frac{12}{8}$$

$$1.5 = 1.5$$

or the equation is in equilbrium.

Example 5

Cross multiplying does not alter an equation and is a method of solving, thus in the above equation multiply the numerators on one side of the equation by the denominators on the other:

$$(2 \times 3) \times (1 \times 8) = (12 \times 1) \times (1 \times 4)$$

or $$48 = 48$$

The equation is still in equilbrium.

Example 6

Solving for an unknown

$4 \times \frac{3}{4} \times A = 8 \times \frac{1}{2}$

Cross multiply

$(4 \times 3) \times A \times 2 = (8 \times 1) \times 4$

$24\,A = 32$

$A = 32/24 = 1\text{-}8/24 = 1\text{-}1/3 = 1.33$

To check the result replace A with the solved value in the equation for simplicity $A = 1\text{-}1/3 = 4/3$

$4 \times \frac{3}{4} \times 4/3 = 8 \times \frac{1}{2}$

$(4 \times 3 \times 4) \times (2) = (8 \times 1) \times (4 \times 3)$

$96 = 96$

DECIMALS

The decimal system is another manner of representing fractions. The following rules must be adhered to when dealing with decimals:

Example 1

Any whole number is not altered by putting a decimal point to the right of it and any number of zeros, thus

4 is equivalent to 4.0000 etc.

Example 2

In adding or subtracting decimals the digits must be kept in their respective positions to the right or left of the decimal point

$$0.12 + 1.0003 = \begin{array}{r} 0.1200 \\ +\underline{1.0003} \\ 1.1203 \end{array}$$

Example 3

In multiplication add the total number of digits to the right of the decimal points and insert in the answer so that there is the same number of digits to the right of the decimal point.

3.43 x 0.5 or

3.43	)-	Total of four digits
.05	)	to right of decimal point
1715		
000		
0.1715	-	Total of four digits to right of decimal point

Example 4

In division multiply the divider by an appropriate multiple of 10 that is, 10, 100, 1000 etc. so that it becomes a whole number. The quotient or number being divided must be multiplied by the same number.

$3.43 \div .05$ or $5 \overline{)343.000}$

$= 68.0$

ANALYSIS BY TERMS

It is important that we use the appropriate physical quantities and convert to a consistent form where necessary. We can only add or subtract numbers having the same physical units, however we can multiply or divide unlike quantities providing we retain the appropriate descriptions. For example:

4 feet - 2 inches should be

48 inches - 2 inches = 46 inches

2 feet x 4 pounds = 8 foot pounds

Vol. = 5 feet x 5 feet x 5 feet = 25 feet feet feet or 25 cubic feet or 25 $ft.^3$

Velocity = distance travelled in a certain time
miles per hour (mph)
feet per minute (ft./min.)

Example 1

Find the rate of discharge through a pipe, given the velocity of flow and the cross sectional area

Let Q = rate of discharge $ft.^3$/min.

V = velocity in ft./min.

A = area in $ft.^2$

Q = V A this is a common hydraulic equation which can be proven by an analysis of terms

$Q = ft.^3/min.$

$V\ A = ft./min. \times ft.^2 = ft.^3/min.$

$\therefore$ the terms are equal on both sides of the equation.

COMMON CALCULATIONS

The following are common calculations carried out in daily plant control.

Example 1

Suspended Solids

Assume 100 mililitres (ml.) of sample

weight of crucible and solids	16.5888 grams
" " "	16.5712 grams
	.0176 grams

since 1 part per million (ppm.) $= \frac{1 \text{ milligram}}{\text{litre}}$

$$A = \frac{.0176 \text{ gr.}}{100 \text{ ml.}} \times \frac{1000 \text{ ml.}}{\text{litre}} \times \frac{1000 \text{ mg.}}{\text{gr.}}$$

cancelling out common terms

$$A = \frac{.0176 \text{ gr.}}{100 \text{ ml.}} \times \frac{1000 \text{ ml.}}{\text{litre}} \times \frac{1000 \text{ mg.}}{\text{gr.}}$$

$$= \frac{176 \text{ mg.}}{\text{litre}} \quad \text{or} \quad 176 \text{ ppm.}$$

The answer suggests a short cut in procedure, however this is the basis of calculation.

Example 2

5-day B.O.D.

Since 1 ml. of 0.025 N Sodium thiosulphate = 0.2 mg. of oxygen, 1 ml. titrated into a 200 ml. sample is equal to 1 ppm. oxygen or 1 ml. titrated into a 100 ml. sample is equal to 0.5 ppm.

Assume that 4.2 ml. of sodium thiosulphate is titrated into 100 ml. of dilution water and that 3.2 ml. is titrated after incubation for 5 days into 100 ml. of a 1% dilution.

ppm. D.O. in diluting water = 4.2 x 2 = 8.4 ppm.

ppm. D.O. in dilution = 3.2 x 2 = 6.4 ppm.

$$\frac{\text{ppm. in dilution water - ppm. D.O. in dilution}}{\text{percent of dilution}} = \text{ppm. 5 Day B.O.D.}$$

$$\frac{8.4 - 6.4}{1\%} = \frac{2}{1/100} = \frac{2 \times 100}{1} = 200 \text{ ppm. 5 Day B.O.D.}$$

CONVERSION FACTORS

Example 1

Million gallons per day (M.G.D.) to cubic feet per sec.

($ft.^3/sec.$)

$$A = 1{,}000{,}000 \ \frac{gal.}{day} \times \frac{1\ day}{24\ hr.} \times \frac{1\ hr.}{60\ min.} \times \frac{1\ min.}{60\ sec.} \times \frac{1\ ft.^3}{6.25\ gal.}$$

cancelling $$A = 1{,}000{,}000 \ \frac{gal.}{day} \times \frac{1\ day}{24\ hr.} \times \frac{1\ hr.}{60\ min.} \times \frac{1\ min.}{60\ sec.} \times \frac{1\ ft.^3}{6.25\ gal.}$$

$$= \frac{1{,}000{,}000\ ft.^3}{24 \times 60 \times 60 \times} 6.25\ sec.$$

$= 1.85\ ft.^3/sec.$ or 1.85 c.f.s.

Example 2

Gallons to cubic feet

1 gallon of water weighs 10 lb.

1 $ft.^3$ of water weighs 62.5 lb.

$\therefore$ 1 $ft.^3$ of water = 6.25 gal.

VOLUMES

Vol. = surface area x depth

Example 1

Tank 20 ft. x 8 ft. x 5 ft.

Volume = 800 $ft.^3$

$$= 800\ ft.^3 \times \frac{6.25\ gal.}{ft.^3}$$

= 5000 gal.

Example 2

Circular tank 20 ft.ϕ x 10 ft. deep.

Surface area of 20 ft.ϕ

$= \frac{\Pi D^2}{4}$

$= \frac{3.14 \times 20^2}{4} = 314 \text{ ft.}^2$

$$\begin{aligned} \text{Volume} &= 314 \text{ ft.}^2 \times 10 \text{ ft.} \\ &= 3140 \text{ ft.}^3 \\ &= 3140 \text{ ft.}^3 \times \frac{6.25 \text{ gal.}}{\text{ft.}^3} \\ &= 19{,}600 \text{ gal.} \end{aligned}$$

VELOCITY

Velocity = rate of travel say ft./sec.

Example 1

An object floating in a channel takes 10 sec. to travel a distance of 5 ft.

$\text{Vel.} = \frac{5 \text{ ft.}}{10 \text{ sec.}} = 0.5 \text{ ft./sec.}$

Example 2

Sewage is entering a plant at the rate of 10 M.G.D.

What is the velocity through a grit channel which is 3 ft. wide and 4 ft. deep?

From before: Q = flow rate

V = velocity

A = area

$Q = V A \text{ or } V = \frac{Q}{A}$

$Q = 10 \text{ M.G.D.} = 10 \times 1.85 = 18.5 \text{ ft.}^3/\text{sec.}$

$A = 3 \times 4 = 12 \text{ ft.}^2$

$V = \frac{18.5 \text{ ft.}^3}{12 \text{ ft.}^2}/\text{sec.} = 1.54 \text{ ft./sec.}$

RATE OF FLOW

Pump capacities can often be calculated by filling or dewatering a tank and observing the change in water depth in a fixed time.

Example 1

A pump lowers the liquid level 1 foot in a tank 40 ft. x 20 ft. in a 30 minute period. What is the capacity (flow rate) of the pump?

From before: $Q = V\ A$

$V = 1\ \text{ft.}/30\ \text{min.}$

$A = 40 \times 20 = 800\ \text{ft.}^2$

$Q = 1\ \text{ft.}/30\ \text{min.} \times 800\ \text{ft.}^2$

$= 800\ \text{ft.}^3/30\ \text{min.}$

$= 26.67\ \text{ft.}^3/\text{min.}$

$= 26.67\ \text{ft.}^3/\text{min.} \times 6.25\ \text{gal./ft.}^3$

$= 167\ \text{gal./min. (G.P.M.)}$

CONCENTRATION IN PARTS PER MILLION (ppm)

The following examples concern the concentrations of solids.

Example 1

100 ml. of raw sewage is filtered through a Gooch crucible. The dry suspended solids weigh 0.0275 gr.

Find total suspended solids in ppm.

1 ppm. = 1 mg./litre or 1 mg./1000 ml.

or 1 mg./1,000,000 mg.

or 1 unit in 1 million units of any kind.

Concentration of dry suspended solids as ppm.

$$= \frac{0.0275\ \text{gr.}}{100\ \text{ml.}} \times \frac{1000\ \text{mg.}}{1\ \text{gr.}} \times \frac{1000\ \text{ml.}}{\text{litre}}$$

= 275 mg./litre or 275 ppm.

Example 2

The Gooch crucible from above is ignited with a loss of weight of 0.0175 grams. Find volatile suspended solids in percent and ppm.

$$\% \text{ volatile solids} = \frac{\text{gr. volatile suspended solids}}{\text{gr. suspended solids}} \times 100$$

$$= \frac{0.0175}{0.0275} \times 100 = 64\%$$

$$\text{Volatile suspended solids} = \frac{0.0175\text{gr.}}{100 \text{ ml.}} \times \frac{1000 \text{ mg.}}{1 \text{ gr.}} \times \frac{1000 \text{ ml.}}{\text{litre}}$$

$$= 175 \text{ ppm.}$$

or Volatile suspended solids = 64% x 275 ppm. = 175 ppm.

Example 3

The flow rate of the above sewage is 2,000,000 G.D. How many pounds of suspended solids received per day?

1 ppm. = 1 lb./1,000,000 lb.

Weight of suspended solids/1,000,000 lb. of sewage = 250 lb.

Number of pounds of sewage/day = 2,000,000 x 10

= 20,000,000

$$\therefore \text{ weight of suspended solids} = \frac{20,000,000}{1,000,000} \times 250$$

= 5000 lb./day

Example 4

A 2% dilution of raw sewage is found to have an oxygen depletion of 4.0 ppm. in 5 days at 20°C. What is the 5 day B.O.D. in ppm., also in lb. when flow is 2,000,000 G.D?

2% dilution = 4 ppm.

then 1% dilution = 4/2 = 2 ppm.

100% dilution = 2 x 100 = 200 ppm.

From before:

$$\text{Weight of 5 day B.O.D.} = \frac{20,000,000}{1,000,000} \times 200$$

= 4000 lb./day

Example 5

A Chlorinator is set at 200 lb./24 hr. at a flow of 2,000,000 G.D. The residual is found to be 0.3 ppm. Find the chlorine demand or chlorine consumed in ppm.

ppm. Cl_2 consumed = ppm. Cl_2 applied

- ppm. Cl_2 residual

Cl_2 applied = $\frac{200 \text{ lb.}}{20,000,000 \text{ lb.}}$ = 10 lb./1,000,000 lb.

or 10 ppm.

Cl_2 consumed = 10 - o.3 = 9.7 ppm.

Example 6

What would the chlorinator feed rate have to be in the above example so that the chlorine residual would be 0.5 ppm?

Cl_2 applied = Cl_2 consumed + Cl_2 residual

= (9.7 + 0.5) ppm.

= 10.2 ppm.

Chlorinator setting = $\frac{10.2 \text{ lb.}}{1,000,000 \text{ lb.}}$ x 20,000,000 lb.

= 204 lb.

SLUDGE VOLUME MOISTURE

Example 1

5000 lb. dry weight of sludge enters a plant per day and 60% is removed. A sludge pump discharges at a rate of 100 G.P.M. and the sludge contains 5% solids. Find the volume of sludge and the length of time required to pump the sludge removed.

Dry wt. of sludge removed = $\frac{60}{100}$ x 5000 = 3000 lb.

5% of wet sludge weighs 3000 lb.
1% " " " 600 lb.
100% " " " 600 x 100 = 60,000 lb.

Vol. of wet sludge = 60,000 lb. x $\frac{1 \text{ gal.}}{6.25 \text{ lb.}}$ = 9600 gal.

Time of pumping = $\frac{9600 \text{ gal.}}{100 \text{ gal./min.}}$ = 96 minutes

Example 2

1000 $ft.^3$ of digested sludge containing 95% moisture is dewatered so that it contains 90% moisture. What vol. does the sludge now occupy? What would the vol. be if the sludge were dewatered to 50% moisture? Note the specific gravity of dry solids and of sludge are both assumed to be 1.0.

Vol. occupied by dry solids (at 5%)

$$= 0.05 \times 1000 = 50 \text{ ft.}^3$$

The same vol. would be occupied by dry solids whether the moisture content is increased or decreased. When moisture = 90%, dry solids = 10%

If 10% of total vol. = 50 $ft.^3$
1% " " " = 5 $ft.^3$
100% or total vol. =500 $ft.^3$

That is sludge with 90% moisture only occupies 500$ft.^3$ whereas 95% moisture sludge occupies 1000 $ft.^3$.

When moisture = 50%, dry solids = 50%

If 50% of total vol. = 50 $ft.^3$
1% " " " = 1 $ft.^3$
100% or total vol. =100 $ft.^3$

That is sludge with 50% moisture only occupies 100 $ft.^3$.

Example 3

A digester produces 10,000 $ft.^3$ of gas per day. Raw sludge is added at the rate of 5000 gal. per day. The raw sludge is 6% solids and is 65% volatile. How many $ft.^3$ of gas are produced per pound of volatile solids added?

5000gal. of sludge per day = 50,000 lb.

Weight of solids added per day = .06 x 50,000

= 3000 lb.

Weight of volatile solids added per day = .65 x 3000

= 1950 lb.

Gas produced per lb. of volatile solids added

$$= \frac{10,000}{1950} = 5.1 \text{ ft.}^3$$

MIXTURES

Example 1

A sewer containing 100 ppm. 5 day B.O.D. discharges into a stream at a rate of 700,000 G.D. Flow in the stream is 20 cfs. and the 5 day B.O.D. of the stream upstream of the effluent is 2 ppm. What is the theoretical 5 day B.O.D. of the mixture below the point of discharge?

5 day B.O.D. of mixture in lb. = 5 day B.O.D. in lb. of effluent + 5 day B.O.D. in lb. of stream.

$$\text{B.O.D. of effluent} = 700{,}000\ \text{G./D.} \times 10\ \text{lb./G} \times \frac{100\ \text{lb.}}{1{,}000{,}000\ \text{lb.}}$$

$$= 700\ \text{lb./day}$$

$$\text{B.O.D. in stream} = 20\ \text{cfs.} \times \frac{1{,}000{,}000\ \text{G./D.}}{1.85\ \text{cfs.}} \times 10\ \text{lb./G} \times \frac{2\ \text{lb.}}{1{,}000{,}000\ \text{lb.}}$$

$$= 216\ \text{lb./day}$$

B.O.D. in mixture = 700 lb. + 216 lb. = 916 lb.

Concentration of B.O.D. in mixture =

$$\frac{916\ \text{lb./D}}{20\ \text{cfs.} \times \dfrac{1{,}000{,}000\ \text{G./D.}}{1.85\ \text{cfs.}} \times 10\ \text{lb./G}}$$

$$= \frac{8.5\ \text{lb.}}{1{,}000{,}000\ \text{lb.}}$$

$$= 8.5\ \text{ppm.}$$

THE SIGNIFICANCE OF

SEWAGE PLANT DISCHARGES ON STREAMS

Part I

CHEMICAL ASPECTS

by

C. E. SIMPSON

Supervisor, Chemical Laboratory

An Address To
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The most damaging materials in sewage are the putrescible organic substances and the suspended solids. In addition there are mineral compounds and those organic substances which resist biological sewage treatment, both of which escape to the stream relatively unchanged from their original form. These will remain in the receiving water in the same way that salt remains in the oceans, and may build up to objectionable levels. For instance, all of the salt spread on roads for snow removal finds its way into our surface waters, either through combined or storm sewers, or by direct drainage. During thaws, this may increase the sodium, chloride, calcium hardness and dissolved solids concentrations in these surface waters well above desirable limits.

An example of synthetic organic materials, many of which are resistant to biological treatment, is synthetic detergents, or 'surfactants', whose foaming effect in the sewage plant and receiving stream you are all probably acquainted with.

The effect of suspended sewage solids is threefold. They impart objectionable turbidity to the water. They settle out in slow flowing sections of the stream to form sludge beds. Unless treatment is complete, they contain a large proportion of organic material which causes putrescence or anaerobic digestion in these sludge beds.

Before we consider the effect of organic material further, let us examine a stream in its normal condition. It may be and probably is, already damaged by silt, suspended soil particles washed from farmer's fields. These cause turbidity and settle out as silt beds, but due to their lower organic content do not putresce as readily as do sewage sludge beds.

The most important condition in a stream is its oxygen balance. Normally, the stream is saturated with dissolved oxygen; it contains as much dissolved oxygen as it can hold in solution. This balance is controlled at the surface of the water. If excess oxygen is present, some is released as bubbles. If insufficient oxygen is present, more dissolves from the air to bring the dissolved oxygen level up to saturation.

There is normally only a slight tendency for stream waters to use up oxygen, due to their low organic content. Most stream waters use up 2 ppm. or less dissolved oxygen during 5 days, some less than 1 ppm. This is easily supplied from the surface, and the stream is maintained in balance, saturated with dissolved oxygen.

The organic substances in sewage discharges tend to rob the receiving water of dissolved oxygen. Natural organic substances in sewage are nutrients, of the same types that humans require (carbohydrates, fats and proteins), the only difference being that in sewage they are already partly decomposed or digested. These organic nutrients are used as food by bacteria; the bacteria require oxygen to metabolize this food. The simple end result is that oxygen is used up in proportion to the amount of organic material. The Biochemical Oxygen Demand (BOD) test measures this tendency of the organic material to rob the receiving stream of dissolved oxygen. Thus the higher the BOD the more oxygen that will be used up, and the more quickly it will be used up.

The effect of the discharge of a sewage on the oxygen balance in a stream can be illustrated by a diagram in which the dissolved oxygen content of the receiving stream, measured at a series of downstream sampling points, is plotted in reference either to mileage downstream, or equally well, to days of flow. The characteristic form of the resulting graph gives it the name "Oxygen Sag Curve". (see Diagrams)

There are three possible forms of the curve. In the first, a small increase in BOD can be met by an equal increase in the rate of aeration in the stream, which replaces the oxygen used up. The result is no change in the dissolved oxygen content, it remains saturated, and no damage results to the stream.

This is the ideal condition to be aimed at in treating the sewage before discharge. No more organic matter, or BOD is discharged than can be taken care of by the stream's aeration capacity, alone.

The discharge of BOD above this capacity results in the second form of the curve. Here the organic materials start using up Dissolved Oxygen faster than aeration can replace it. The Dissolved Oxygen level falls. As the Biochemical Oxygen Demand is met, the remaining demand grows less. Finally a point is reached where aeration can meet the demand, then more than meet it. Thus the Dissolved Oxygen level reaches a minimum, then recovers to its original level of saturation. (see figure 2)

The third and most severe condition results from discharging BOD above both the stream's capacity for aeration, and its reserves of dissolved oxygen. In this case, the Dissolved Oxygen level is reduced to zero. The entire stream becomes septic. Anaerobic bacterial action takes place and continues until the organic material is reduced to a point where the aeration rate, which has continued at its maximum, finally is able to cope with the remaining BOD. The Dissolved Oxygen content then recovers gradually to a saturated level.

The end-products of aerobic and anaerobic decomposition of organic matter are almost entirely different, in the stream as well as in your aeration tanks and digestors.

Decomposition End Products of Elements Present in Organic Matter

	Aerobic (using dissolved oxygen)		Anaerobic (avoiding using dissolved oxygen which is absent or scant)
(Carbon)	C--> /--> CO_2	(Carbon) (dioxide)	C--> /--> CH_4 (methane), some CO_2
(Hydrogen)	H--> /--> H_2O	(water)	H--> /--> H_2 (gaseous hydrogen)
(Sulphur)	S--> /--> SO_4	(sulphate)	S--> /--> H_2S (sulphides)
(Nitrogen)	N--> /--> NO_3	(nitrate)	N--> /--> NH_2, N_2 (amines, gaseous nitrogen)
(Phosphorous)	P--> /--> PO_4	(phosphate)	P--> / ? (not known)

/ = many intervening steps. (Aerobic)

/ = many intervening steps. (Anaerobic)

The anaerobic intermediate and end products are characteristically offensive in appearance and odour. Note the substitution of hydrogen in these end-products, in place of oxygen which is lacking.

The aerobic end products are relatively inoffensive and act as fertilizers. (see figure 4).

These idealized graphs skip over a number of pertinent factors.

Aeration rate - The maximum aeration rate is not constant throughout a stream. The surface/volume ratio varies. Thus a narrow, deep section of the stream has less surface from which to obtain oxygen than a shallow, wide, stream section. Diffusion is a negligible factor in dispersing the oxygen picked up at the surface throughout the depth of a stream. The most important factor of all is turbulence, as it is in your aeration tanks. A rapidly flowing turbulent section of a stream has many times the aeration capacity of a quiet sluggish section. Thus in any stream the recovery graph will vary with topography. Rapidly flowing shallow streams that may show no immediate effect, although loaded with sewage BOD may show distinct depletions or even septicity, in calm deep pools or impoundments some miles downstream from the source of pollution.

The stream may also be aerobic at the surface and still show oxygen depletions or septicity at or near the bottom, especially where there are sludge deposits. (see figure 5).

The most critical conditions below a discharge of organic pollution occur during the summer. A number of factors are responsible, all of which reinforce one another. First, stream flows are at minimum; there is less dilution for a given quality and volume of sewage discharge. The resulting BOD concentration in the stream will be higher. Temperatures are higher. This has a threefold effect. The Biochemical Oxygen Demand is exerted more rapidly due to the increased rate of bacterial metabolism. The saturation level for Dissolved Oxygen is lower, since as the temperature rises, less oxygen is held in solution. At higher temperatures, fish become more susceptible to damage from low oxygen levels. With lower flows, streams are sluggish and aeration capacity from turbulence may decrease.

The total end result is that any adverse condition in the stream is intensified profoundly during the summer.

The cards are not all stacked against you however. The same acceleration of biological processes occurs in your plant, and by promoting this through efficient control, you should be able to achieve treatment adequate to maintain your receiving water in good condition in the summer as well as throughout the year.

SUMMARY:

The most objectionable materials in domestic sewage are the organic substances which tend to rob a receiving stream of Dissolved Oxygen. Depending on the amount discharged, that is, on the efficiency of treatment, the stream may:

- suffer no damage,

- may have dissolved oxygen levels depleted to levels hazardous or injurious to fish and other organisms, or

- may be rendered totally septic.

These conditions increase in severity during the summer due to a number of natural factors, including temperature, stream flow and reaeration capacity. Thus particularly efficient treatment is required throughout this period.

THE SIGNIFICANCE OF

SEWAGE PLANT DISCHARGES ON STREAMS

PART II

BACTERIOLOGICAL ASPECTS

by

L. T. VLASSOFF

Bacteriologist

An Address To
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Intermediate Sewage Works Operators' Course
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March 5, 1962

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INTRODUCTION

In a previous discussion attention was called to certain basic principles underlaying sewage treatment (stabilization, or mineralization). You will recall the method by which bacteria act on sewage. Under aerobic conditions they attack the organic waste directly using oxygen and forming cell-body-building materials, gases (carbon dioxide (CO_2), etc.), waste products and energy (for example heat). The important point is that oxygen is used. Once all of the oxygen (O_2) is depleted, anaerobic conditions set in catering to another group of bacteria present in sewage and in the stream which continue to decompose sewage, but are not able to mineralize it completely. These anaerobic organisms produce unstable malodorous products such as methane, hydrogen sulfide (H_2S), ammonia (NH_3), etc., and release energy in the process. Numbers of aerobic organisms decrease rapidly under these conditions. Depending upon available oxygen, numbers of predators and other antagonistic forces, and the quantity and type of food present in the waste, some bacterial groups will exist in large quantities, others in trace amounts and some will remain undetectable by available identification techniques. (Graph number 4).

The reactions that occur in the ideal stream from the time sewage is dumped (day and mile 0) until the stream recovers (day 9 and mile 108) occur in four zones. These are sometimes named:

1) Zone of degradation
2) Zone of active decomposition
3) Zone of recovery
4) Zone of clear water

This discussion will deal mainly with the first and second of these where the predominating microscopic organisms are bacteria and fungi.

Zone of Degradation

The Zone of Degradation is characterized by the presence of large quantities of undecomposed or slightly altered constituents of sewage, ample oxygen in the upper portion facilitating aerobic bacterial life, and turbid water hindering growth of algae. Fecal bacteria are numerous, indicated by coliform populations over 10,000 per ml., with an initially low number of microscopic animals (protozoans).

The varied aerobic bacterial population increases rapidly in proportion to the food available and continues until the organic waste and oxygen supply become limiting. This increase was discussed by Mr. G. Kay in the first course, where he pointed out the features of the logarithmic bacterial growth rate curve. (See graph number 4).

Rapid decomposition sets in and continues until oxygen and food decrease sufficiently to become limiting (approximately four days). Coincident with decomposition, carbon dioxide content increases and larger plants and animals disappear. Microscopic animals, notably ciliated protozoans, increase and may reach populations of several thousands per milliliter.(See graph number 6.) Here they feed on the growing bacterial population until they fall victim to rotifers and crustaceans, the larger microscopic animals.

The efficiency of the sewage-consuming biological machine depends on a close-knit savage society in which one organism captures and eats another. Bacteria do a more rapid job of breaking down sewage when several species are present and actively multiplying. However, stabilization is increased markedly when ferocious bacteria-eating ciliates are introduced, keeping the bacterial population actively multiplying. The role of protozoans in the decomposition of sewage is not completely defined, but it is known that they may also feed on sewage particles as well as on bacteria.

At the same time that this activity is progressing in the aerobic layer, an equally active anaerobic microbial population is attacking the oxygen-depleted settled sludge, over which a "sewage fungal" carpet is growing. The term "sewage-fungi" is applied to any visible, filamentous, thread-like growth observed in sewage polluted water that does not possess a green colour. (A greyish, cotton-like mass of threads attached to rocks, limbs or to any debris). It varies greatly in composition depending on environment and may be composed of certain of the filamentous bacteria, filamentous algae and true fungi. (See Graph number 5)

Of the bacteria, <u>Beggiatoa</u> and at times <u>Leptothrix</u> and <u>Crenothrix</u> are encountered. The true fungi are represented by filaments of <u>Leptomitus</u>, <u>Achyla</u> and <u>Saprolegnia</u>, the latter often found growing on dead animals rather than plant tissue.

Oscillitoria, a blue-green filamentous algae and Chara and Spirogyra, grass green filamentous algae, are occasionally found in material called "sewage fungi". Their green colour may be masked by encrusted slimes and they are frequently mistaken for fungi.

These "fungi" rely almost entirely on organic matter for nutrients and are most efficient in decomposing some of the more difficult-to-break-down materials such as cellulose and waxes. As an example of the simple needs of fungi, mildew (caused by a group of fungi) can grow readily on damp leather shoes where nutrients are at a minimum and where even bacteria cannot exist. These "fungi" are generally aerobic and will not be active when the oxygen level is low. (See graph number 5).

At the sewage discharge they will increase rapidly forming the first large population of organisms to attack the sewage in the stream. Most of the organic matter they break down is utilized to build cell "tissue", releasing by-products in a form easily decomposable by the next dominant population - the bacteria. The mass of filaments left behind after conditions become unfavorable for fungal growth due mainly to a lack of oxygen, are also easily digested by bacteria. (See graph number 4)

The zone of degradation persists until the oxygen falls to a level of about 3.5 ppm. Then the zone of active decomposition commences.

Zone of Active Decomposition:

This zone continues until the oxygen (O_2) drops to a minimum and again returns to 40% of saturation (3.5 ppm. O_2). It is characterized by a greyish colour at first, replaced by an almost black colour in the septic area where the black slime is replacing the "sewage fungi" and ammonia (NH_3) and hydrogen sulfide (H_2S) and other foul smelling organics are increasingly evident.

The aerobic bacterial activity has gradually been replaced by an active population of anaerobes. The coliform content here may be between 10 to 1,000 per ml. Toward the end of this zone bacterial activity decreases markedly, due to antagonistic elements (limited food and increase in number of protozoans), and is replaced by that of larger forms and algae. (See graph number 6).

In the Zone of Recovery and Zone of Cleaner Water, bacteria and molds are virtually out of the picture. Here organisms higher in the food chain - the so-called "higher forms of life" predominate.

SUMMARY

1) "Sewage fungi" form the first dominant group of the food chain replaced by

2) Aerobic bacteria and protozoans, gradually substituted by

3) Anaerobic bacteria and more protozoans as oxygen becomes limiting,

4) Soon greatly reduced in numbers by large populations of protozoans followed by retifers and crustaceans and finally

5) By the so-called "higher forms of life".

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PART III

THE BIOTA

by

JOHN H. NEIL

Biologist

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Part III

THE BIOTA

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JOHN H. NEIL

Biologist

The first life to develop in the stream in response to a waste discharge has been described as untold billions of bacteria and millions of small one celled animals called protozoans which feed on both the bacteria and the organic matter. It would be reasonable to suppose that as soon as conditions improved slightly that more competition for the large quantity of organic food and more predators on this large population of small animals would develop. This does in fact happen and large numbers of rotifers and crustaceans as are illustrated in Fig. 6 develop.

There are a few larger organisms who by virtue of some special adaptation are soon able to move into the areas of active decomposition. These are illustrated in the active decomposition portion of the stream illustrated in Fig. 7 & 8. The special adaptations are interesting. The grub with the tube extending to the surface is called the rattailed maggot. It survives in an environment often having little or no oxygen by breathing air through the tube. Slightly further downstream sludge worms appear often in sufficient numbers to turn the bottom of the stream completely red. These worms are specially adapted by having body fluid containing a substance like haemoglobin in our blood that has a great affinity for oxygen. The other red larvae is an immature stage of mosquite-like insect that also has a red body fluid.

Early in the recovery stage more forms begin to appear. The sow bug and the leech are depicted here and like the earlier forms they too come in considerable numbers. For instance, it is not uncommon to find a dozen leeches under every stone in the recovery area. As the stream recovers more and more species appear, dragonfly nymphs become common and later the sensative Mayflies and Damsel-flies appear. The hardy fish move up and down this section feeding when conditions are good and moving out when the oxygen drops. Usually these are coarse fish like carp and minnows and it is not until the stream is almost fully recovered that the desirable game species return. In the case illustrated, in the graphs, this occurs about 90 miles downstream.

The discussion thus far has not included any mention of the plants. In the previous discussion, the fate of nitrogenous substances was described. Anybody who has farmed or grown a garden knows that nitrogen and phosphorous are a great stimulus to plants. Sewage is rich in both of these so it is not surprising that they show a similar response in water. In Fig. 5 the areas of active decomposition contain mostly sewage moulds as the waste products of decomposition are poisonous to all but the most hardy forms. However, as soon as these toxic substances have dissipated and the nitrites changed to nitrates a very favourable environment has been developed similar to an oxidation pond and the algae multiply enormously. This heavy growth of algae may be of several kinds as you proceed downstream and may also include heavy growths of rooted aquatic plants. Probably the stream will never be the same but the heaviest growth will take place in the upper stream before the fertility is all used up. This algae and other plant growth can in itself be an undesirable effect of sewage discharge.

The discussion thus far has dealt with the worst condition. We hope that with an efficient well-run sewage treatment plant the receiving stream will never reach this deplorable condition.

The activated sludge process is a biological one which takes the place of the worst condition in the stream. The initial growth of bacteria and protozoans has taken place and broken down most of the organic materials. The nitrogen has been changed from an organic form through ammonia and nitrites and to relatively stable nitrate form. In other words, it has replaced about the first five or six days in the theoretical river. At this stage the rotifers and crustaceans can live to consume the bacteria and other forms responsible for the previous treatment. The substances that were toxic to the algae have disappeared and the nutrients are in a form suitable for immediate use. Unfortunately, the water is not yet suitable for all forms of life but at least the fish and varied aquatic life are only about three days away and not nine as would be the case with no treatment.

This example shows a condition where relatively poor dilution occurs. Many plants have much better dilution and with the aid of good treatment the clean water environment can exist right up to the point of discharge of the well treated effluent.

In concluding it is essential to point out that the biota of the stream receiving the waste from your treatment plant will not reflect the average condition of your effluent but the worst condition that has occurred in perhaps the previous year. The life in the stream must continue to breathe just as people, and while it can be said that in

a year people breathe on an average of thirty times a minute this is not of much importance if for 10 minutes during the year there was no air to breathe.

With this thought in mind, therefore, every precaution should be taken to prevent by-passing, pumping of sludge, over chlorination or any other operation that will cause a critical stream condition. This is of particular importance in the warm water season when available dilution is at a minimum and the quantity of oxygen in the water at a minimum. When shutdowns are necessary, try to schedule them for spring or wet weather or even at night when the sewage strength is weak. Use the remainder of the plant rather than just letting it go raw. Remember that your good work for a year previous can be wasted when one batch of raw sewage or sludge escapes proper treatment.

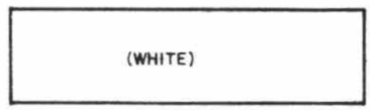
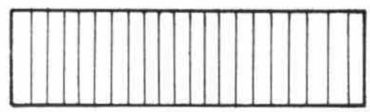
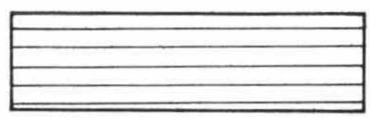
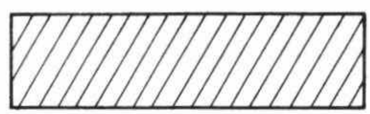
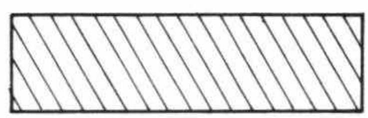
LEGEND FOR DIAGRAMS FIGURE 1-8 AS REPRODUCED FROM THE PUBLIC WORKS MAGAZINE FOR JULY 1959 FROM THE ARTICLE:

"STREAM LIFE AND THE POLLUTION ENVIRONMENT"

BY

A.F. BARTSCH, ASSISTANT CHIEF FOR SPECIAL TECHNICAL SERVICES

W.M. INGRAM, IN CHARGE, BIOLOGICAL FIELD INVESTIGATIONS FOR WATER POLLUTION CONTROL, WATER SUPPLY AND WATER POLLUTION RESEARCH, R.A.TAFT SANITARY ENGINEERING CENTER, U.S. PUBLIC HEALTH SERVICE, CINCINNATI, OHIO.

COLOURS IN ORIGINAL DIAGRAM	TRANSFERRED TO BLACK HATCHING IN REPRODUCED DIAGRAM
GREEN	
ORANGE & YELLOW	
RED	
BLUE	
GREY	
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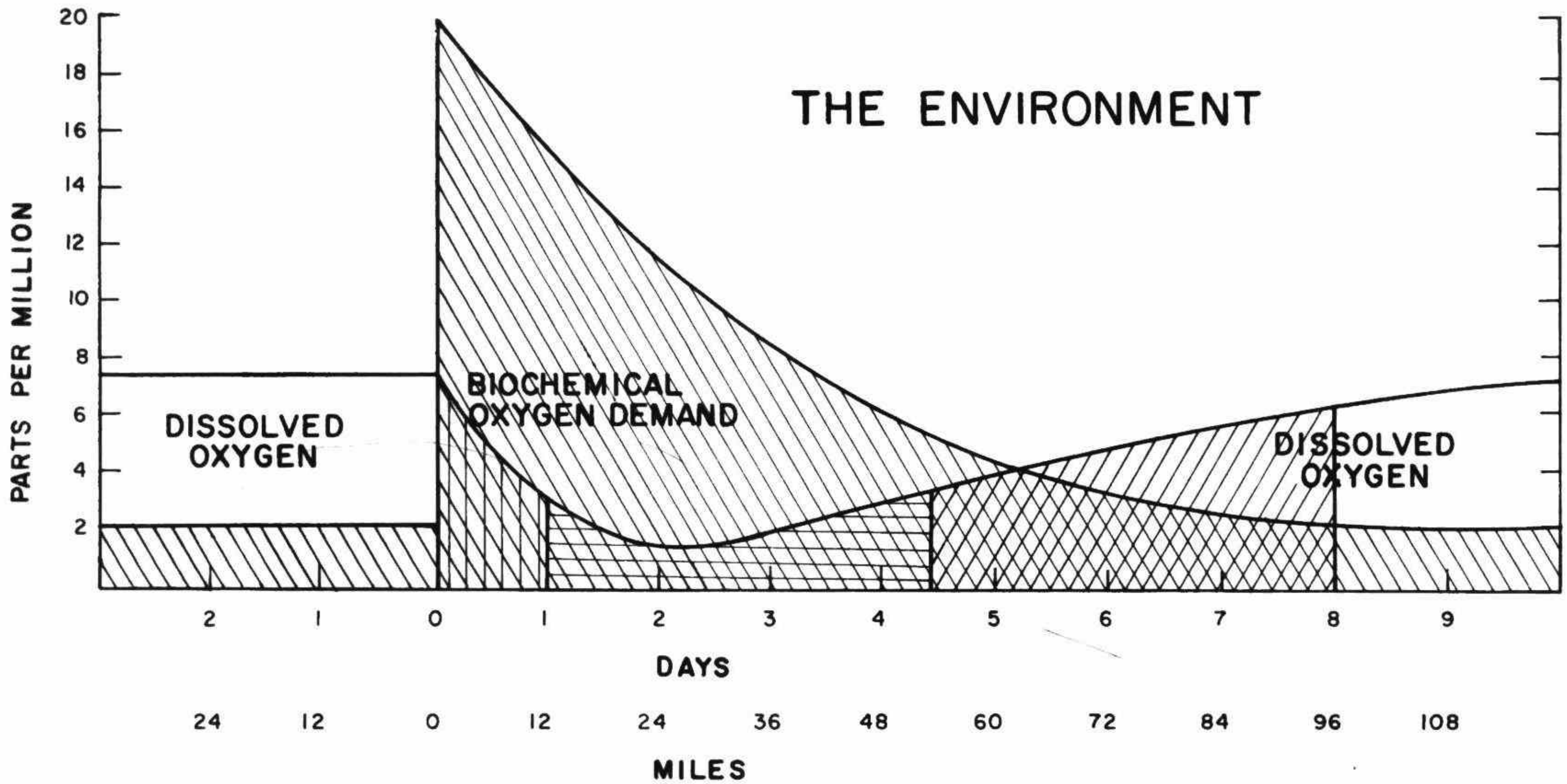

FIG. I THE ASSUMPTIONS IN THE HYPOTHETICAL POLLUTION CASE UNDER DISCUSSION ARE A STREAM FLOW OF 100 cfs, A DISCHARGE OF RAW SEWAGE FROM A COMMUNITY OF 40.000, AND A WATER TEMPERATURE OF 25° C, WITH TYPICAL VARIATION OF DISSOLVED OXYGEN AND B.O.D.

J.R.

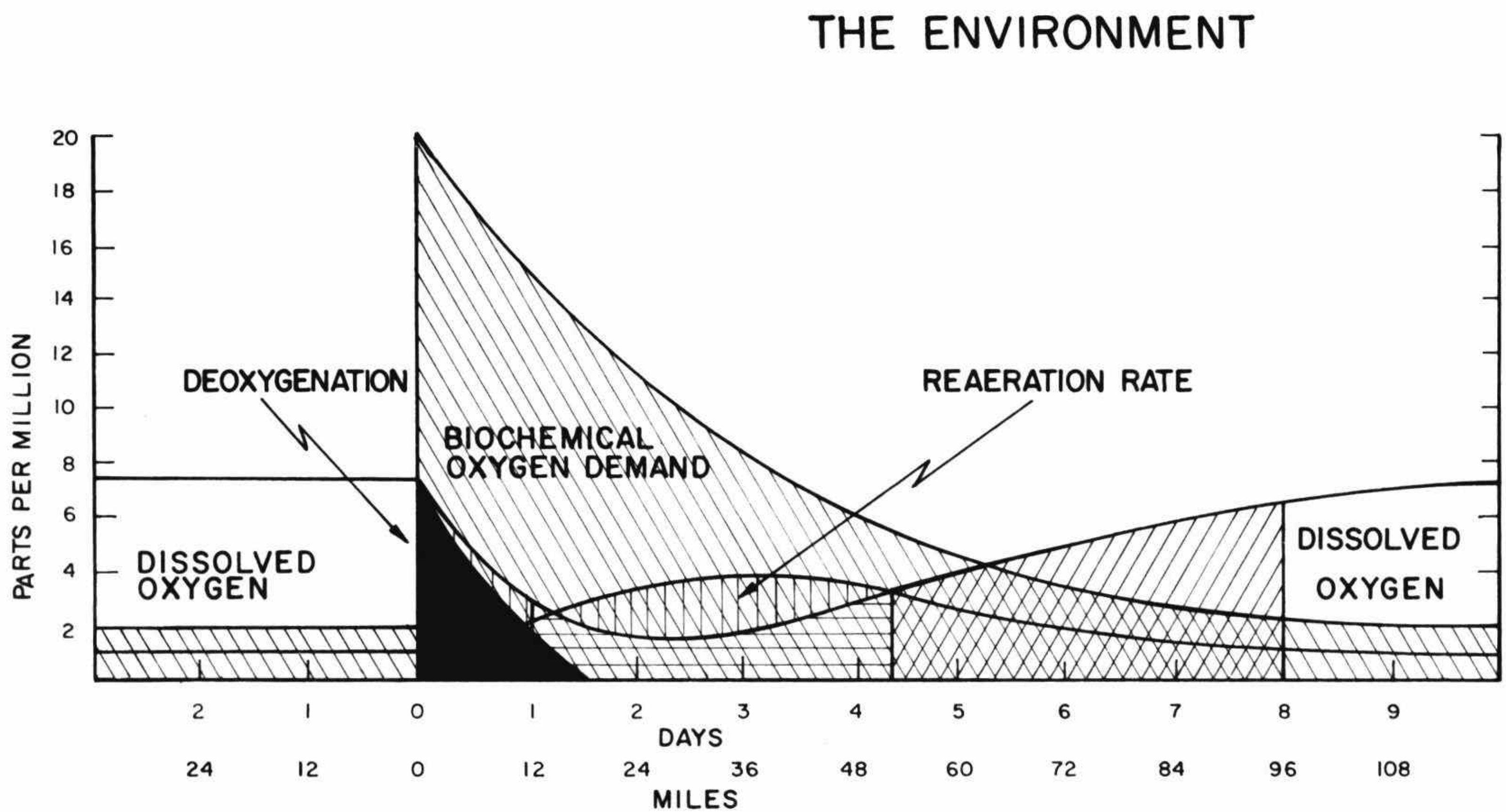

FIG. 2 THE DISSOLVED OXYGEN CONCENTRATION IN THE STREAM IS PARTIALLY DESTROYED BY THE POLLUTION LOAD. FULL DEPLETION IS AVOIDED BY REAERATION PROCESSES.

J.R.

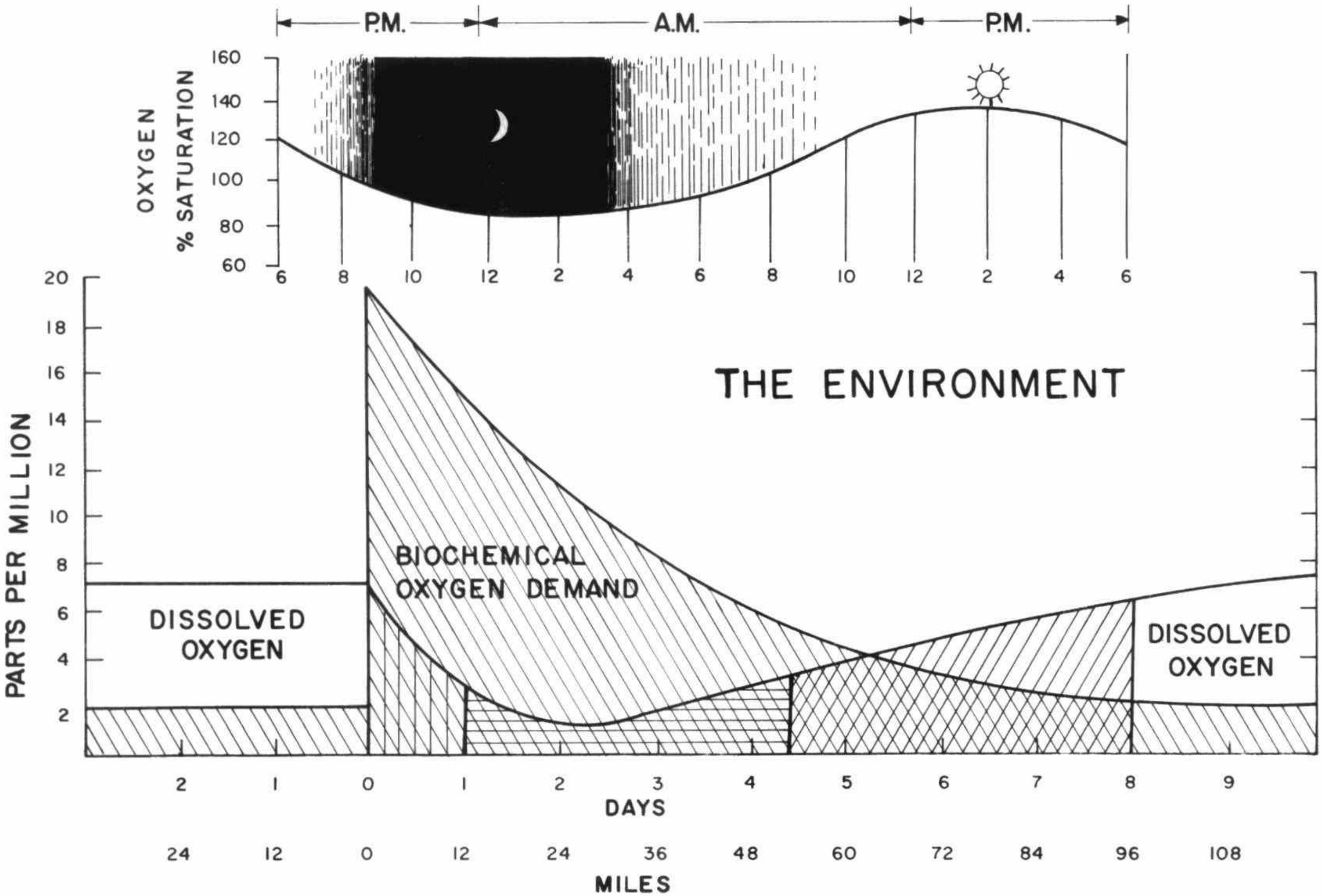

FIG. 3 DISSOLVED OXYGEN FLUCTUATES ACCORDING TO AVAILABLE LIGHT, A RESULT OF PHOTOSYNTHESIS. THUS, VALUES ON THE LOWER CURVE ARE SUBJECT TO DAILY VARIATION.

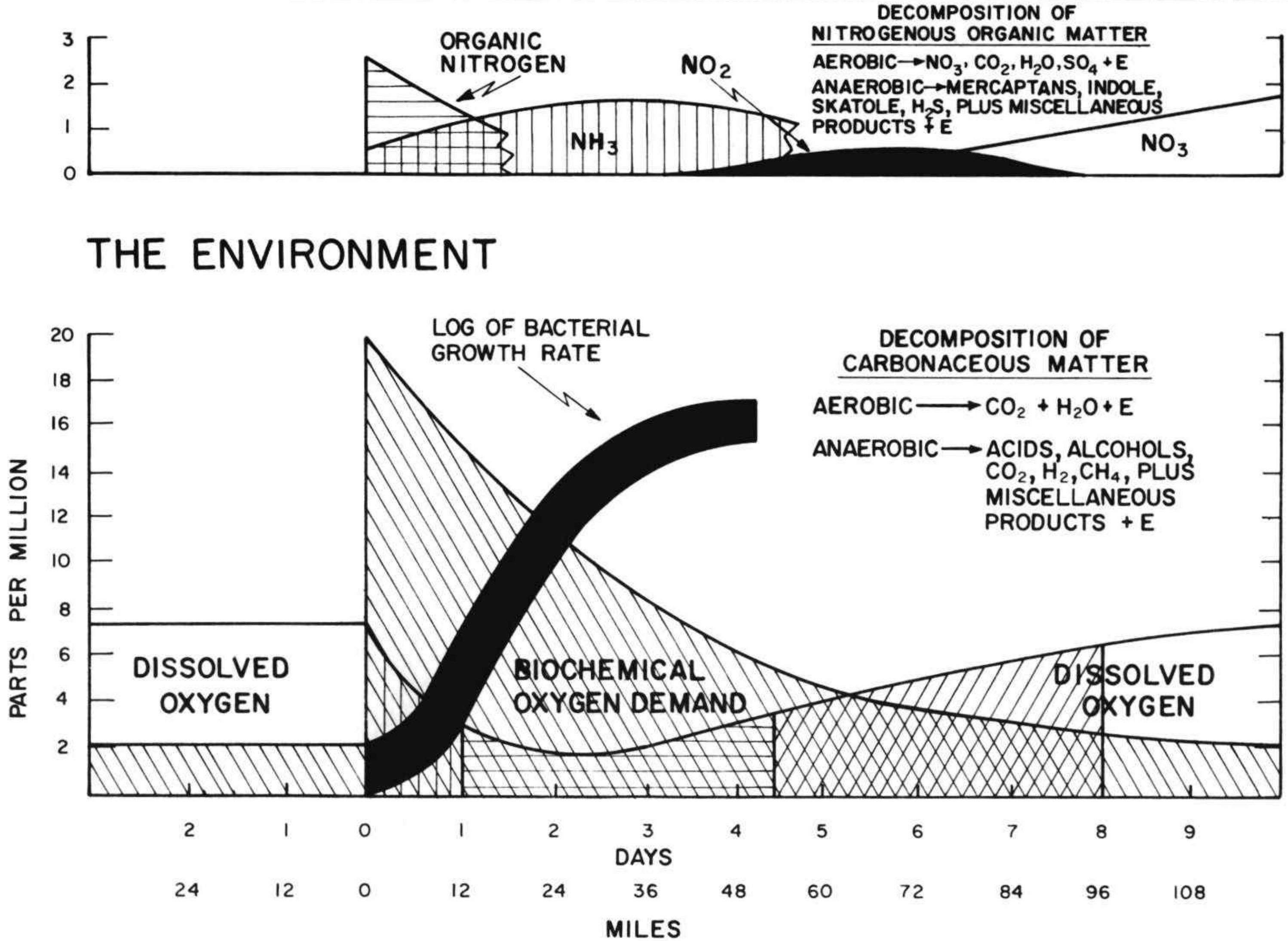

FIG. 4 WITH A HEAVY INFLUX OF NITROGEN AND CARBON COMPOUNDS FROM SEWAGE, THE BACTERIAL GROWTH RATE IS ACCELERATED AND DISSOLVED OXYGEN IS UTILIZED FOR OXIDATION OF THESE COMPOUNDS. AS THIS PROCEEDS, FOOD IS 'USED UP' AND THE B.O.D. DECLINES.

THE BIOTA

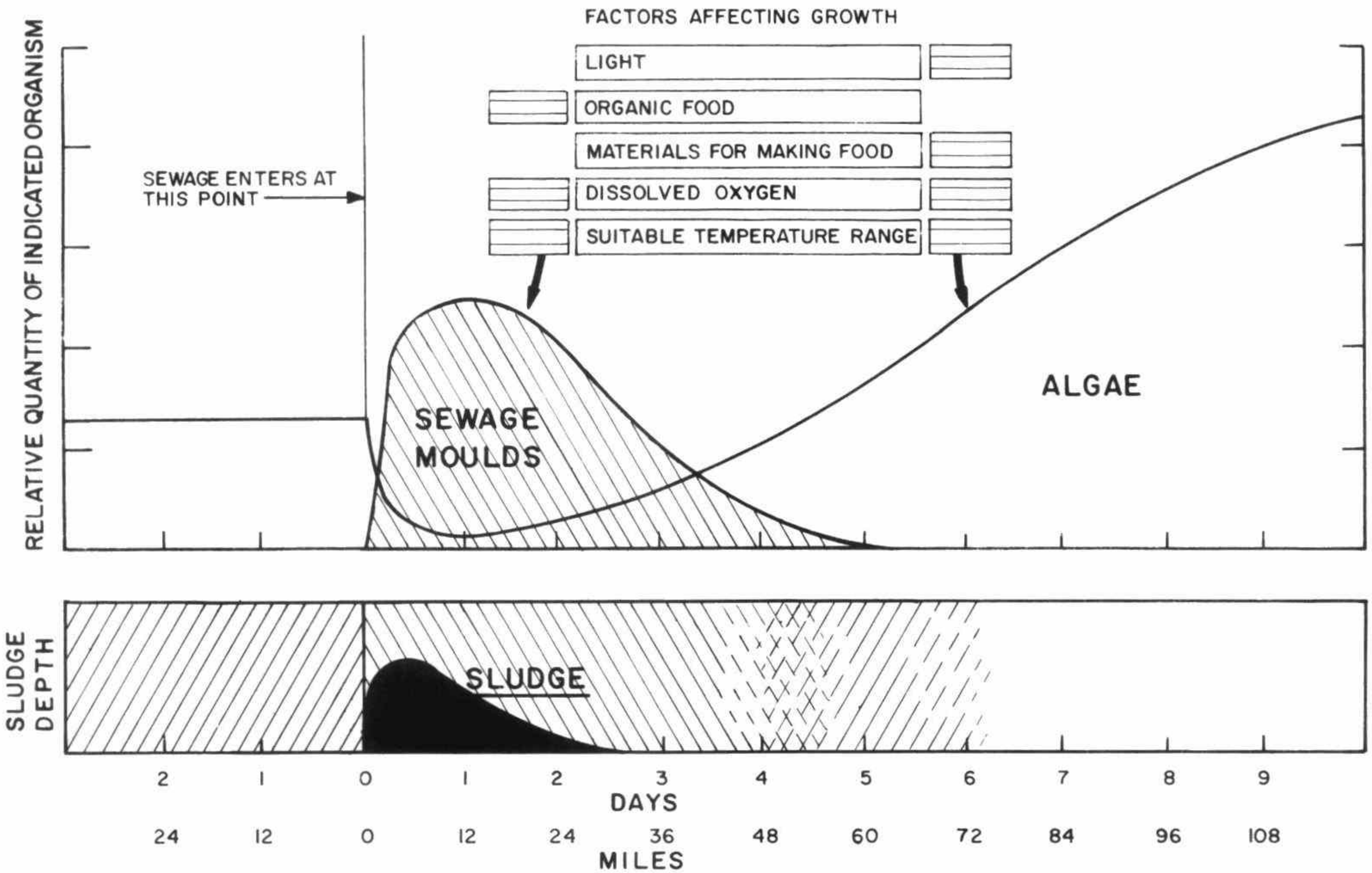

FIG. 5 SHORTLY AFTER SEWAGE DISCHARGE, THE MOULDS ATTAIN MAXIMUM GROWTH. THESE ARE ASSOCIATED WITH SLUDGE DEPOSITION SHOWN IN THE LOWER CURVE. THE SLUDGE IS DECOMPOSED GRADUALLY; AS CONDITIONS CLEAR UP, ALGAE GAIN A FOOTHOLD AND MULTIPLY.

J.R.

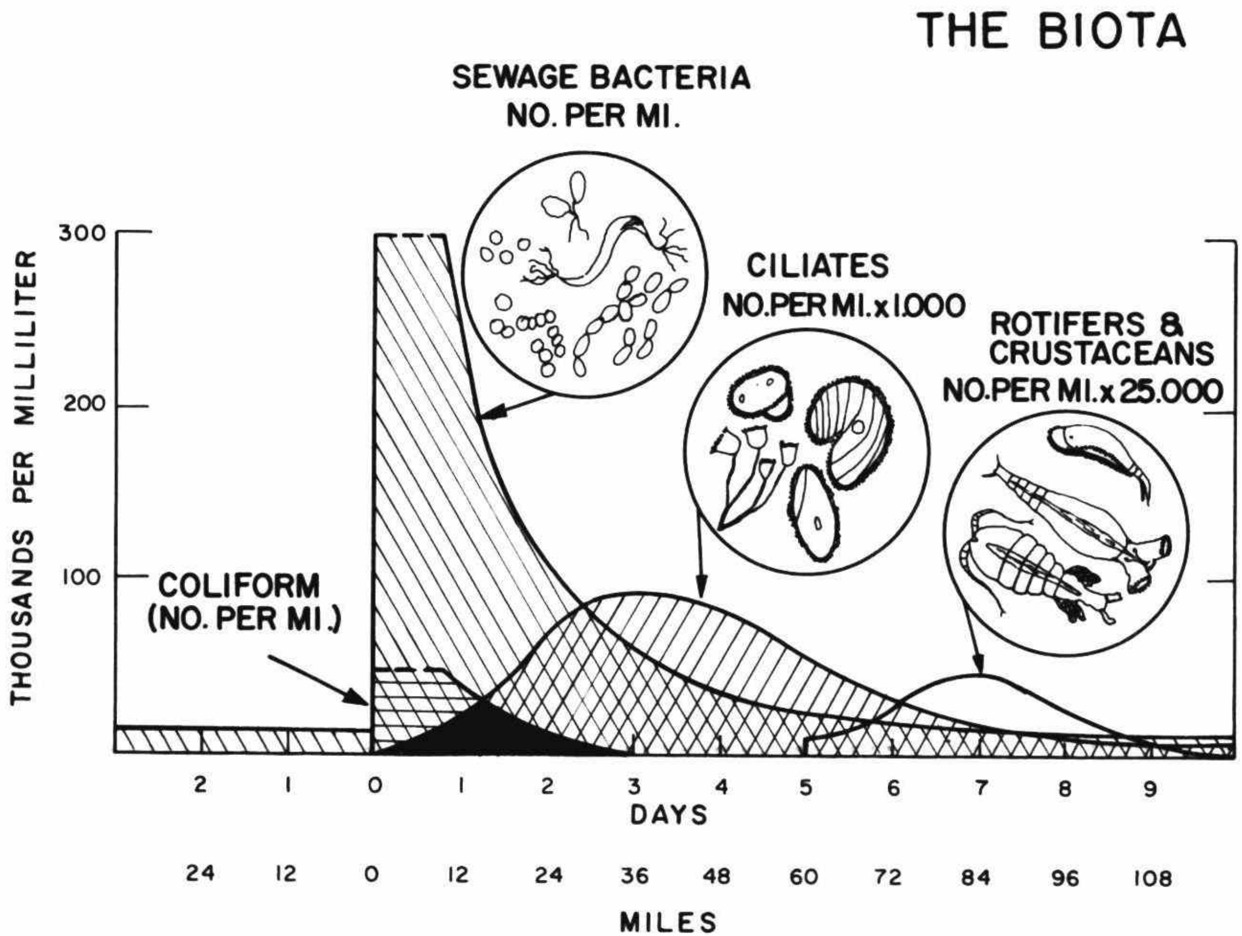

FIG. 6 BACTERIA THRIVE AND FINALLY BECOME PREY OF THE CILIATES, WHICH IN TURN ARE FOOD FOR THE ROTIFIERS AND CRUSTACEANS.

J.R.

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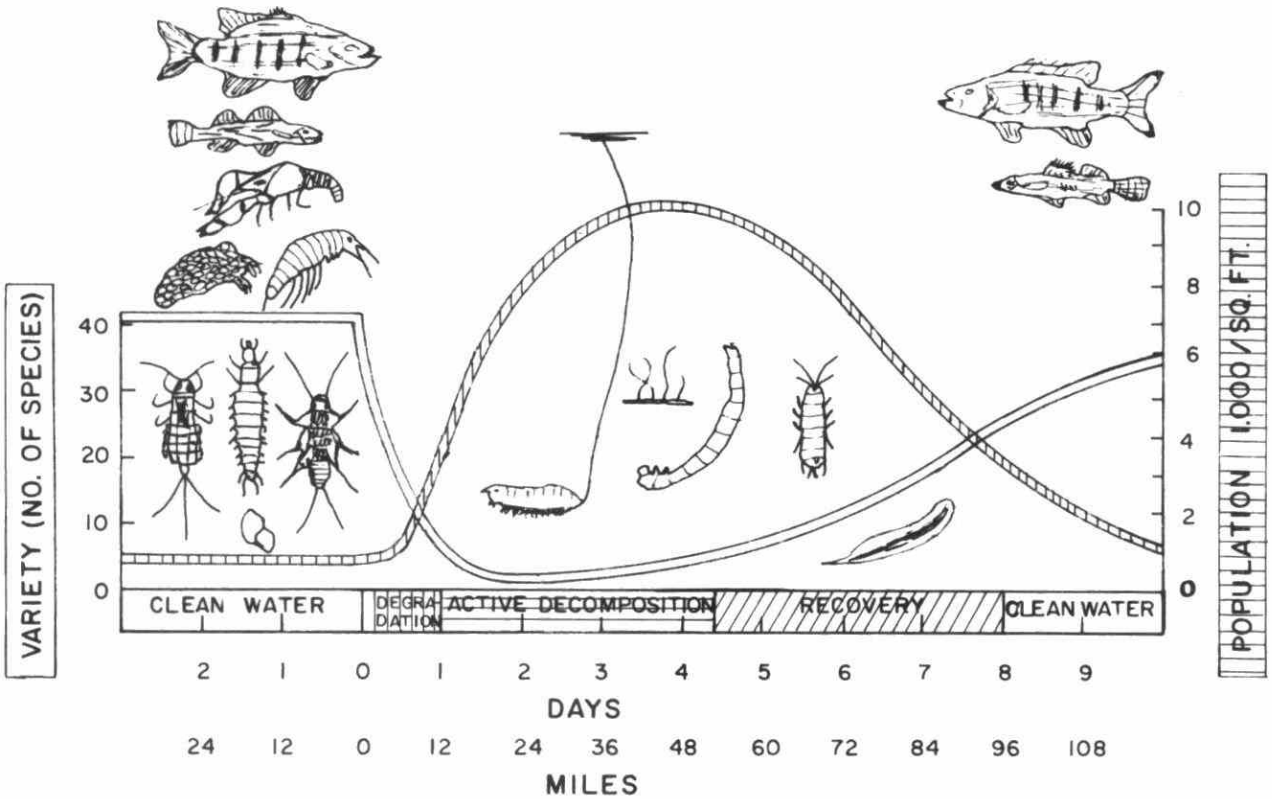


FIG. 7 THE UPPER CURVE SHOWS THE FLUCTUATIONS IN NUMBERS OF SPECIES; THE LOWER CURVE, THE VARIATIONS IN NUMBERS OF EACH.

J.R.

THE BIOTA

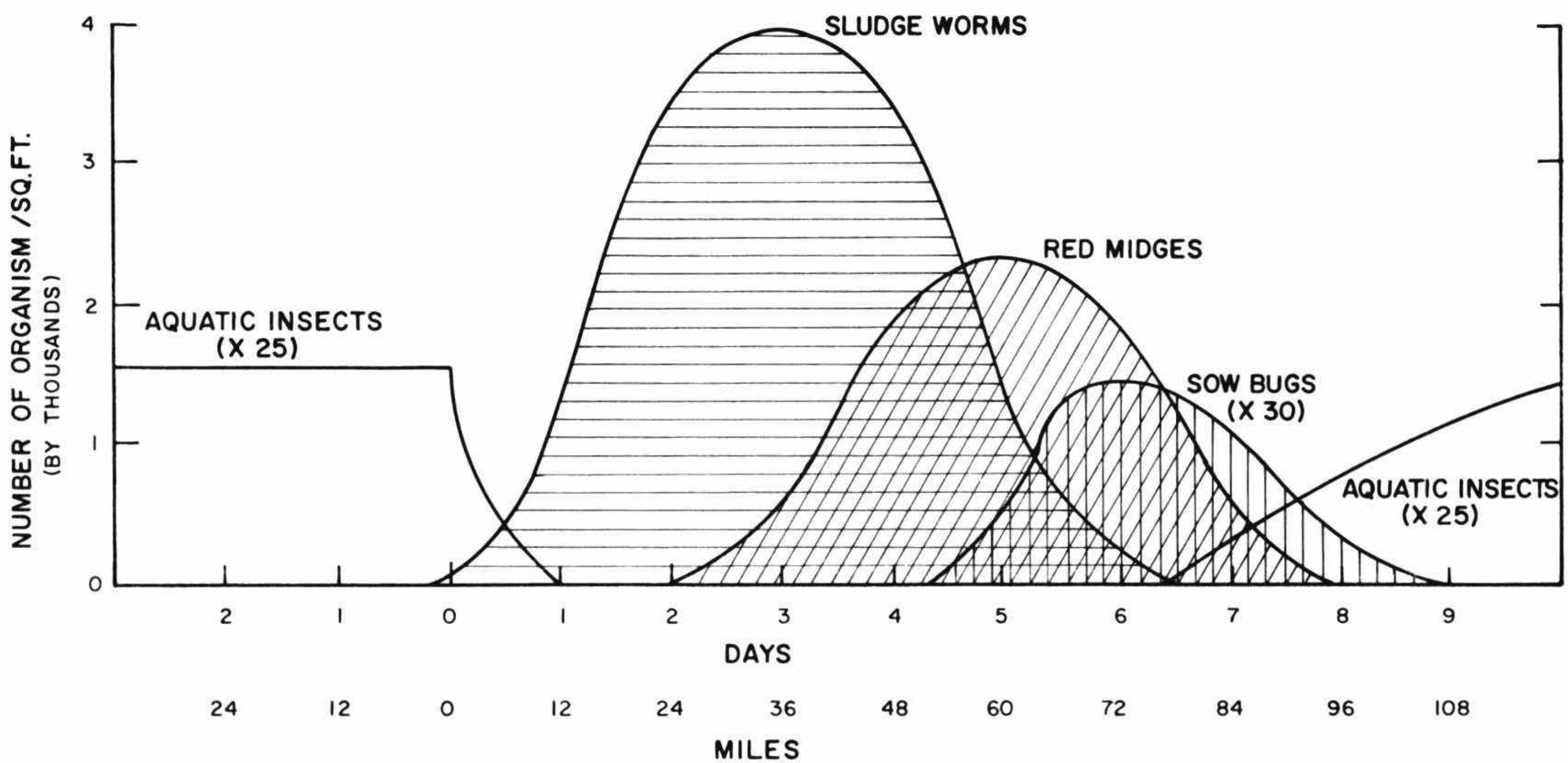


FIG. 8 THE POPULATION CURVE OF FIGURE 7 IS COMPOSED OF A SERIES OF MAXIMA FOR INDIVIDUAL SPECIES, EACH MULTIPLYING AND DYING OFF AS STREAM CONDITIONS VARY.

J.R.

RAW SEWAGE CHARACTERISTICS

by

K. SHIKAZE

Chemical Engineer

An Address To
The Ontario Water Resources Commission
Intermediate Sewage Works Operators' Course
Toronto, Ontario
March 5, 1962

RAW SEWAGE CHARACTERISTICS

by

K. SHIKAZE

Chemical Engineer

If I were to ask each of you who are present here today to describe in a few words the raw sewage characteristics at your sewage plant, I would probably get as many different answers as there are sewage plants represented here. For this reason, I cannot hope to cover all of the problems which you may have encountered in your raw sewage. However, what I will try to do is to describe in general terms, some of the raw sewage characteristics and then cite some variations which will probably apply to some of you.

SOURCES

Sewage is the waste water of community life. It originates from many sources. The most important from the public health point of view are human and animal wastes. I think that I can safely say that the most important or perhaps most troublesome from the sewage plant operator's point of view are the industrial wastes. Sewage also receives household wastes from kitchen and laundry. In addition, street washings and storm flows enter the sewers and alter both the volume and composition of sewage. In many instances, sewers are laid below the ground water level and without completely tight joints thus permitting the infiltration of ground water, thereby affecting the volume and composition of sewage. Industries must also dispose of their wastes and the most convenient method is discharge into the sewers. These wastes vary widely in type and in volume. Most operators at one time or another have experienced or will experience some operational problems in their plant which can be directly attributed to industrial wastes. Some of the specific industrial waste problems which may be encountered will be dealt with in another lecture.

APPEARANCE

Sewage has been defined as a liquid of widely varying composition with an unpredictable and often unsatisfactory behaviour. Ordinary fresh domestic sewage is grey in colour and has the odour and appearance somewhat like soapy dish water. It is somewhat turbid

and will contain certain floating matter such as matches, bits of paper, soap, feces, rags, garbage, oily patches and numerous other materials. The liquid portion of the sewage will contain materials in solution which of course cannot be seen and millions of bacteria and other microscopic organisms. Stale or septic sewage is darker in colour, sometimes black and with varying intensity of disagreeable odours. The floating materials are much more difficult to distinguish and their appearance may be quite altered from the original. Gas bubbles will be seen rising to the surface.

The change from a fresh to a septic sewage is brought by the bacteria present in the sewage. These bacteria also effect changes in the dissolved materials present in the original sewage and the intensity of their action is governed by certain factors. For example, flat grades in sewers will cause the sewage to flow slower and allow a longer time for the bacteria to act. Also, long trunk sewers will cause a similar situation. Low spots and dead ends will act as reservoirs or pools with a similar result. High temperatures within certain limits also cause the bacteria to act faster, while the lower temperatures have the opposite effect. Unventilated sewers are also a factor. By excluding oxygen or air, better environments are set up, allowing the bacteria to act faster.

QUANTITY OF SEWAGE

The quantity of sewage per capita varies within wide ranges. In cities with separate sewers, the amount may vary from 40 to 280 gallons. The probable average is about 100 gallons. At first it would seem that the flow of sewage from a city should be about the same as the water consumed. This is not strictly true as the sewage may vary from 70 to 130% of the water consumption. Where large amounts of water are used for sprinkling it is obvious that it will not reach the sewers, while on the other hand many cities have industrial plants which derive their water from other sources other then the city supply and then discharge their wastes into the city sewers.

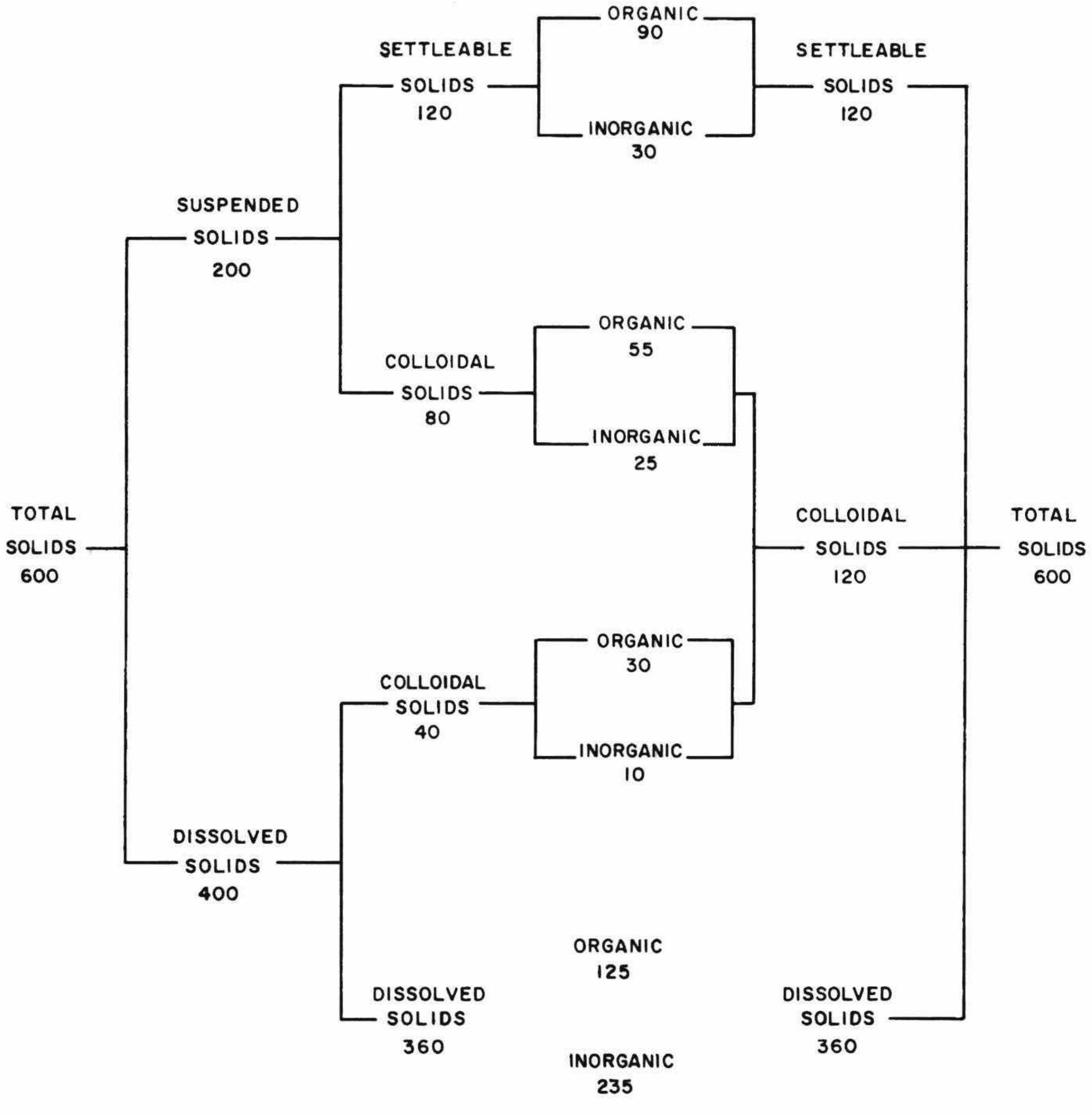
CHEMICAL COMPOSITION

Chemically, sewage is made up of a great many materials which are either suspended or dissolved in water. The water supply itself, contains certain materials dissolved in it, usually minerals of a harmless nature and these minerals along with the other materials that find their way into the sewage constitute what is known as total solids, which as stated previously, are both dissolved and suspended or floating. This solid material in the sewage is divided into two main parts; first, the relatively organic substances that are the waste products of living animal and plant organisms and of their activities in our community life and second, the relatively inert inorganic compounds. I might add here, that the solids constitute less than 0.1 of 1% of the total weight of the sewage. The more than 99.9% of water provides volume and a means of carrying the solids, however, the solids are the important part of a problem which the treatment and

FIGURE I.

PHYSICAL CONDITION AND COMPOSITION OF SOLIDS IN AN AVERAGE DOMESTIC SEWAGE

(Numbers are in parts per million)



disposal of municipal wastes presents. It may be said that sewage is 99.9% water and 0.1% impurities making it a much purer substance than our usual standard of purity of 99 and 44 one hundredth percent (99.44%).

ORGANIC SOLIDS

Organic solids are generally of animal or vegetable origin including the waste products of animal and vegetable life, dead animal matter, plant tissues or organisms but may include synthetic organic compounds. They are substances which contain, hydrogen and oxygen, some of which may be combined with nitrogen, sulphur or phosphorous. The principle groups are proteins, carbohydrates and fats together with the products of their decomposition through the activity of bacteria and other living organisms and they are combustible, that is they can be burnt.

INORGANIC SOLIDS

Inorganic solids are those substances which are inert and are not usually subject to decay. They are frequently designated as mineral substances and include sand, gravel, silt and mineral salts in the water supply which produce the hardness and mineral content of the water. In general, they are non-combustible.

The amount of these solids, both organic and inorganic in sewage imparts to it what is frequently termed its strength. Actually, the amount or concentration of the organic solids in their capacity to undergo decay or decomposition is a principal part of this strength. The greater the concentration of organic solids, the stronger the sewage. Thus strong sewage can be defined as one containing a large amount of solids and in particular, organic solids. A weak sewage is one containing only a small amount of organic solids. Solids can also be grouped depending on their physical state as suspended solids, colloidal solids, and dissolved solids, each of which may include both organic and inorganic solids.

SUSPENDED SOLIDS

Suspended solids are those which are visible in suspension in the water. They are the solids which can be removed from the sewage by physical or mechanical means, such as sedimentation or filtration. Settleable solids are that portion of the suspended solids which are of sufficient size and weight to settle in a given period of time, usually one hour . They are usually reported in millilitres of solids per litre of sewage. They are about 75% organic.

COLLOIDAL SUSPENDED SOLIDS

These are somewhat loosely defined as the difference between the total suspended solids and the settleable solids. They constitute that portion of the total suspended solids about 40%, which are not readily removed by physical or mechanical treatment.

DISSOLVED SOLIDS

The term dissolved solids as commonly used in discussing sewage is not technically correct. All of these solids are not a true solution but they include some solids in the colloidal stage. Dissolved solids as a whole are about 40% organic and 60% inorganic.

TOTAL SOLIDS

Total solids as the term implies includes all of the solids constituents of sewage. They are the total of the organic and inorganic solids or the total of the suspended and dissolved solids. In an average domestic sewage, they are about half organic and half inorganic and about two thirds in solution and about one third in suspension. It is the organic half of the solids which are subject to decay and constitute the main problem in sewage treatment. Figure 1 which shows the wastes of the various types of solids as carried through the definitions is based on an average domestic sewage equivalent to approximately 100 gallons per capita per day. Addition of storm water flows or ground water infiltration may change these solids relationships markedly. Similarly, the introduction of industrial wastes may increase the solids content particularly the organic solids with very definite variations in the strength of the sewage. Also, sewage varies widely in both composition and volume from hour to hour depending upon changes in community activity. Obviously, sewage is likely to be at its maximun strength and flow during the day time and be at a minimum during the night hours. Also, sewage may vary in its composition from day to day with corresponding changes in industrial and community activities from which the sewage originates. On Sundays and weekends and holidays flows and strength are frequently reduced because of the lowered rate of communal activity. Any table of sewage composition can give only an average composition, the amount of the solids indicated can not be applied equally to all sewages at all times.

DISSOLVED GASES

Sewage contains small and varying concentrations of dissolved gases. Among the most important of these, is oxygen present in the original water supply and also dissolved from air in contact with the surface of flowing sewage. This oxygen is familiarly known as dissolved oxygen. In addition to dissolved oxygen, sewage may contain other gases such as carbon dioxide

FIGURE 2.

CARBON CYCLE

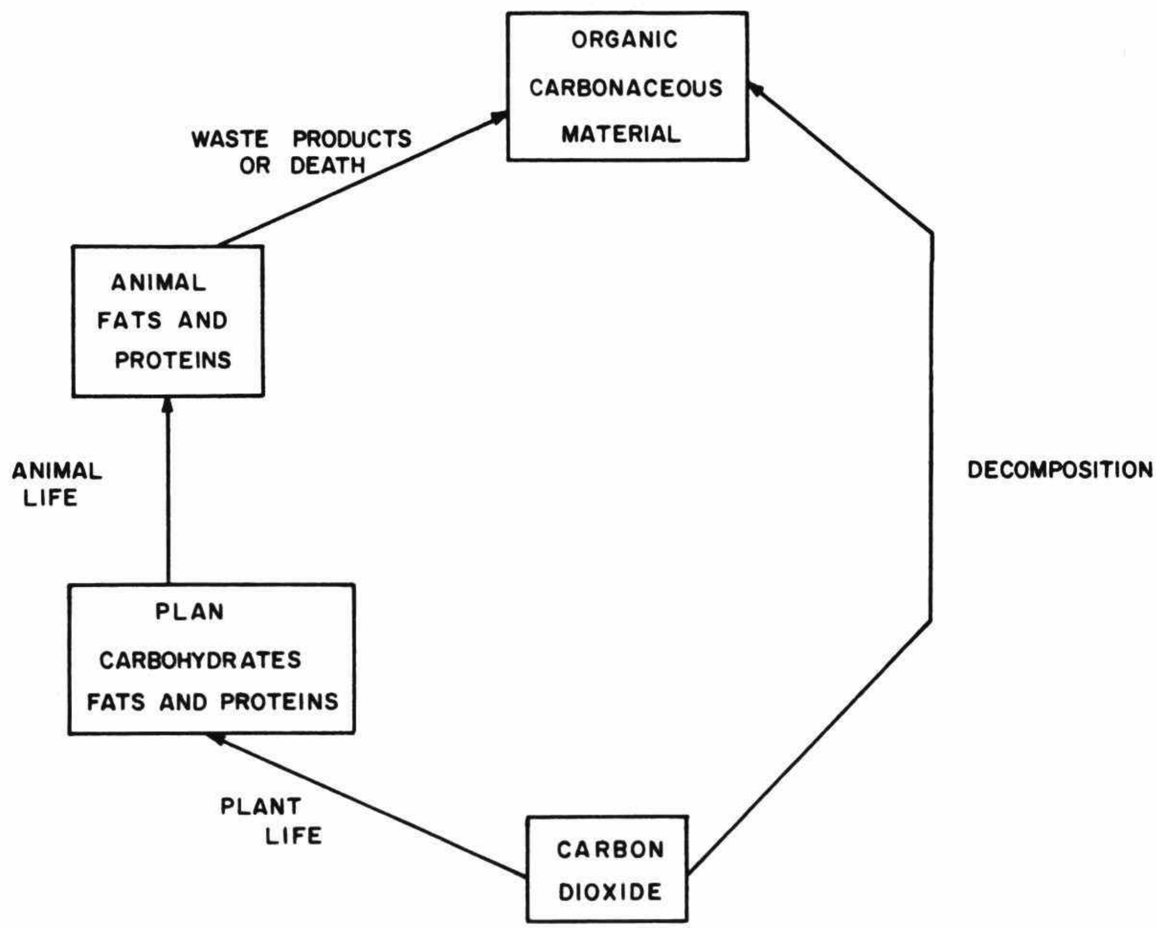


FIGURE 3.

NITROGEN CYCLE

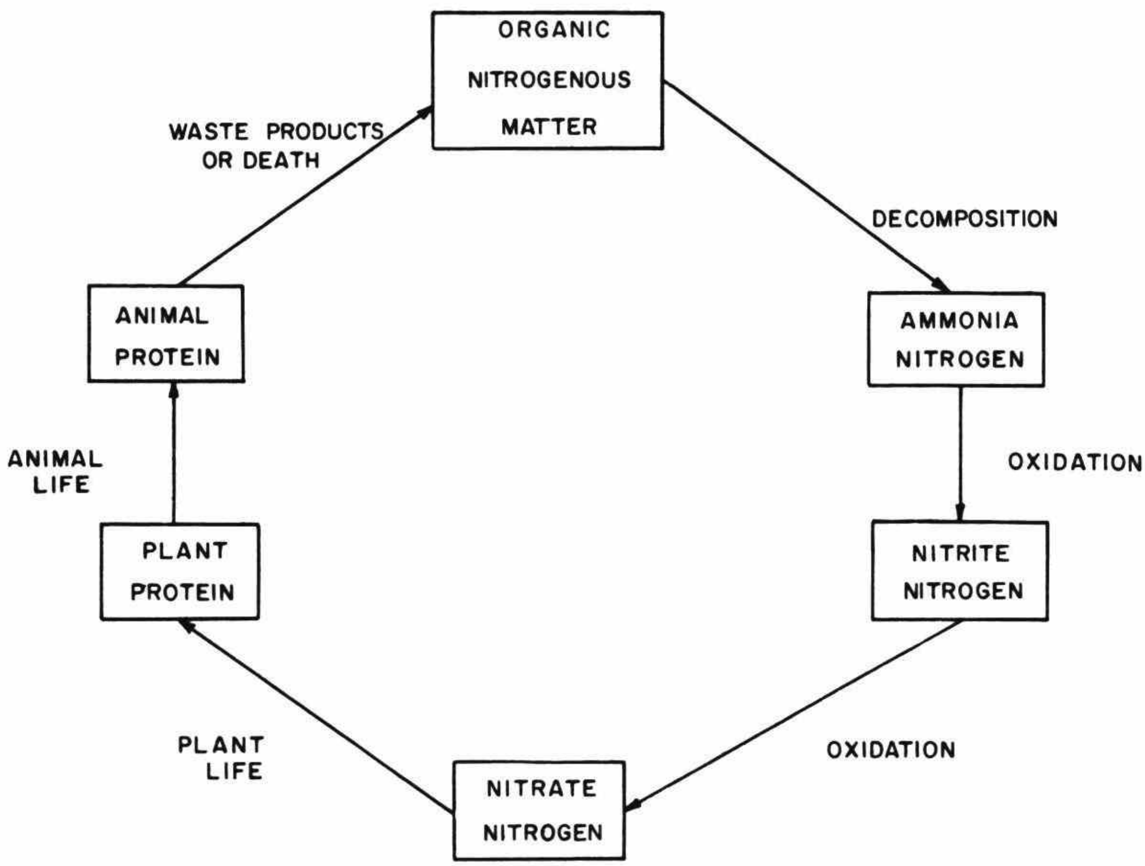
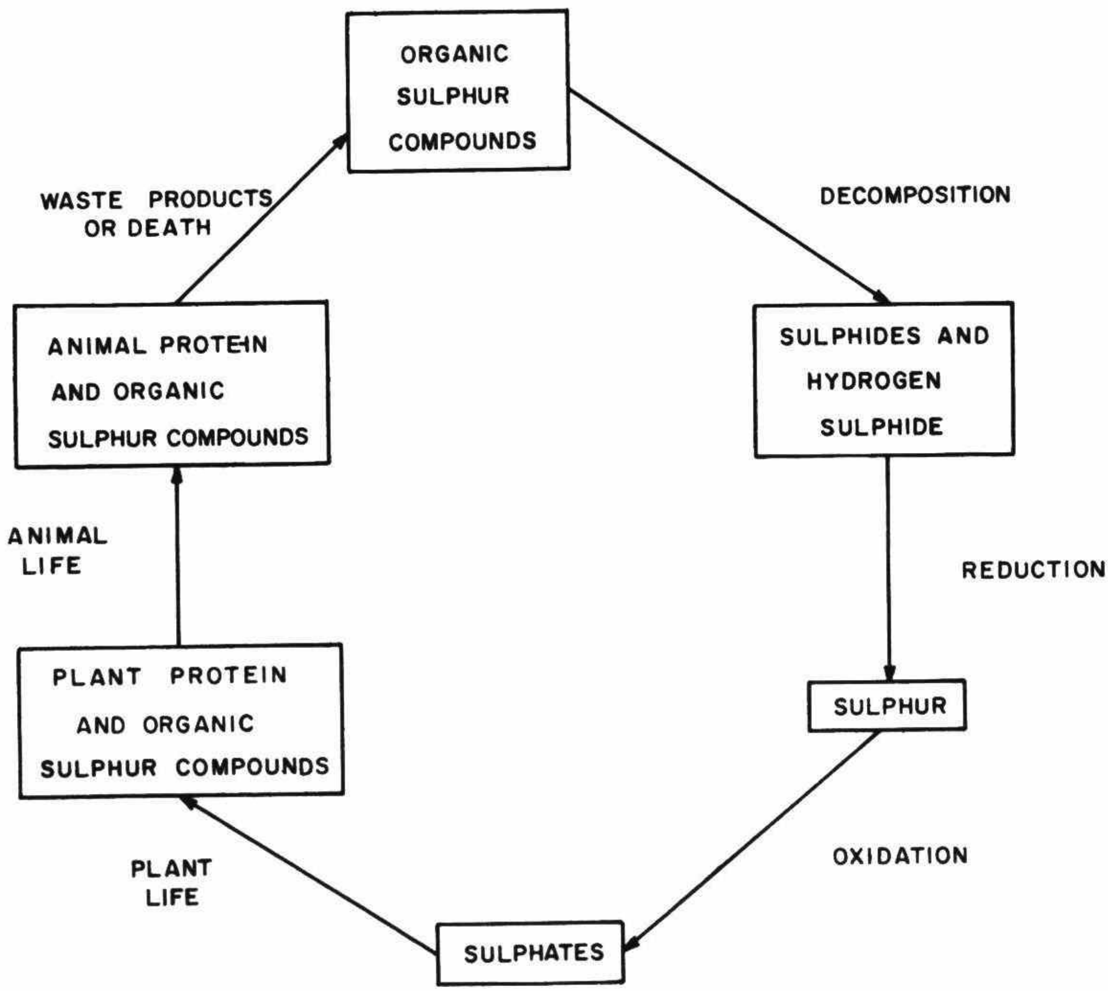


FIGURE 4.

SULPHER CYCLE



resulting from a decomposition of organic matter, nitrogen dissolved from the atmosphere, hydrogen sulphide formed by the decomposition of organic and certain inorganic sulphur compounds. These gases, although small in amount, function importantly in the decomposition and treatment of sewage solids and signify to a major degree the progress of such treatment procedures.

SAMPLING

Two distinct types of sampling may be used. Catch or grab samples and integrated or composite samples. Because of continual variations in the character of sewage, particularly raw sewage, grab samples are not significant as they represent conditions only at the time of sampling. The use of such samples should be avoided when possible. However, bacteriological examination and the determination of dissolved gases require the use of grab samples. They should be collected with great care and with sufficient frequency so that the tabulated results will give a correct average of the material sampled.

Composite samples are prepared by mixing measured portions of the sewage which have been collected at frequent intervals. These are more satisfactory then grab samples, since, if properly collected and composited, they represent the average character of the sewage over a long period of time. To provide composite samples representative of the sewage, the individual proportions should consist of volumes proportional to the flow at the moment of collection, thus measurement of sewage flow is an important component of sampling. If the quantity of sewage at the plant is not known and cannot be neasured then the individual samples should be of equal volume. I would assume that most of the operators here to-day collect their composite samples in this matter.

Composite samples at hourly intervals are usually satisfactory, however, a shorter time, if feasible, would be better and would be better and would be more representative samples. Under certain conditions, as when wide variations in composition occur suddenly and irregularly a shorter time interval between samples may be imperative. Occasionally, longer intervals may be unavoidable but they should never exceed four hours. When personnel is available, samples should be composited over a period of 24 hours. Shorter periods, which are necessary in small plants without night operation, would fail to include the effect of night flows which are usually less concentrated than those during the day. For a routine control of sewage treatment operations, particularly in those plants where attendance at night is limited or not available, valuable information can be obtained by the examination of samples composited over an eight hour period. These should consist of portions collected at least at hourly intervals and the eight hour period should cover those hours of the day which include the heaviest flow. For example, sampling should be started at 8 a.m. and continue until 4 p.m. and under ordinary conditions the heaviest flow which comes at 10 a.m. is represented as our composite sample. For proper sampling procedure, the initial sampling from treatment units can then be delayed in accordance with the time of detention of the sewage in the plant. In this way, samples of the raw sewage and of

the various effluents will be comparable and that they will represent so far as possible the same sewage.

CYCLES OF DECAY

One of the first things the study of chemistry teaches is that matter can not be destroyed, but it can be changed from one form to another. This is particularly emphasized in sewage treatment. The knowledge of the carbon and nitrogen cycles of life, death and decay, is therefore of utmost importance. Since sulphur compounds are usually considered the chief source of odours the sulphur cycle is also important. These cycles are shown graphically and should be self-explanatory. They are shown in figures 2, 3 and 4.

NUTRIENTS IN SEWAGE

The nutrients which are of primary concern to an operator of an activated sludge sewage treatment plant are nitrogen and phosphorus. The two elements are considered to be the limiting factors in the development of a good biological population. It has been generally concluded that a nitrogen-to -BOD ratio of 1-to20 and a phosphorus-to-BOD ratio of 1-to-100 by weight is necessary for optimum biological growth. Domestic sewage normally contains an excess of both nitrogen and phosphorus. However, if an industrial waste entering the sewage plant has high volume and high strength and is deficient in either phosphorus or nitrogen, then operational problems will arise because the domestic sewage may not contain enough nutrients to make up the deficiency. If a deficiency of nutrients does occur, it may become necessary to add some nitrogen or phosphorus to provide optimun conditions for the activated sludge.

BIOLOGICAL COMPOSITION OF SEWAGE

Sewage also contains countless numbers of living organisms, most of them too small in size to be visible except with the use of a microscope. They are a natural living part of the organic matter found in sewage and they are important because they are one of the reasons for sewage treatment and the success of the treatment.

The microscopic living organisms may be considered to be of two general types; bacteria and other more complex living organisms.

BACTERIA

Bacteria are living organisms, microscopic in size, which consist of a single cell and are similar in functions and life processes to plants. Some bacteria are motile, able to move about freely by their own power and others are non-motile. Like all living organisms, bacteria require food, oxygen and water. Bacteria can be divided into two main groups; Parasitic bacteria and Saprophytic bacteria.

Parasitic bacteria are those which normally live off of another living organism, known as the host, since they require a food supply already prepared for their consumption. In general, they do not develop outside the body of the host. Parasitic bacteria, generally, originate in the intestinal tracts of humans and animals and reach the sewage by means of body discharges. Pathogenic bacteria are specific types of parasitic bacteria which produce toxic or poisonous compounds that cause disease in the host. These specific types may be present in sewage receiving the body discharges of persons infected with such diseases as typhoid fever, dysentery, cholera or other intestinal diseases. The possible presence and potential danger of these micro-organisms in sewage is one of the principal reasons why sewage must be carefully collected, adequately treated and disposed of safely.

Saprophytic bacteria are those which feed on dead organic matter. They decompose organic solids to obtain their nourishment and produce waste substances which consist of both organic and inorganic solids. It is this activity of decomposition which is of utmost importance in sewage treatment methods designed to facilitate or hasten natural decomposition of organic solids in sewage. There are many species of saprophytic bacteria, each of which plays a specific role in the breakdown of organic solids in sewage. Each species tends to die away following completion of its part in the process of decomposition.

All of the bacteria, parasitic and saprophytic, require in addition to food, oxygen for respiration. The dissolved oxygen in water is utilized. These organisms are known as aerobic bacteria and the process of degradation of organic solids which they carry out is aerobic decomposition, oxidation or decay. The decomposition proceeds without foul odours or unsightly conditions.

Anaerobic bacteria are those which cannot exist in the presence of dissolved oxygen and they obtain their oxygen from the oxygen content of organic solids and some inorganic solids, which is made available by the decomposition of these solids.

To complicate the reactions involved in the decay of organic matter, certain aerobic types can adjust themselves to live and grow in the presence of dissolved oxygen and are thus termed facultative anaerobic bacteria.

Conversely, some types of anaerobic bacteria can become accustomed to live and grow in the presence of dissolved oxygen and are thus termed facultative anaerobic bacteria.

When the environmental conditions of food supply, oxygen, moisture and temperature are maintained at their optimum amounts for the full functioning of bacteria, then, the decomposition of sewage solids and the sewage treatment processes proceed in a natural orderly manner.

VIRUSES

There is one other form of life found in sewage that is of interest to the sewage treatment plant operator. These are viruses. They are smaller than bacteria and generally too small to be seen underthe ordinary microscope used in bacteriological work. They do not play any significant part in sewage treatment plant processes, but they are important in that they are, like pathogenic bacteria, the causative agent of a number of diseases in man. For example, the virus of hepatitis orginates in the intestines of man and are carried with intestinal wastes to the sewage.

VARIOUS METHODS OF GRIT REMOVAL

AND HANDLING OF SCREENINGS

by

P. G. COCKBURN

Assistant District Engineer

An Address To
The Ontario Water Resources Commission
Intermediate Sewage Works Operators' Course
Toronto, Ontario
March 5, 1962

VARIOUS METHODS OF GRIT REMOVAL AND HANDLING OF SCREENINGS

by

P. G. COCKBURN

Assistant District Engineer

INTRODUCTION

This lecture is a continuation of the lecture given in the Basic Course entitled Preliminary & Primary Treatment at Sewage Treatment Plants and, as a basis for the subsequent discussions, reference should be made to the sections in that lecture pertaining to screening and grit removal facilities.

QUANTITY OF SCREENINGS

A coarse screen (2 inches or greater) will usually collect about one cubic foot of solids per million gallons of sewage. This material will consist of dead animals, wood, rags, and paper.

Medium screens have openings ranging from 0.5 to 1.5 inches and will ordinarily collect 5 to 15 cubic feet of material per million gallons of sewage. These screenings usually contain considerable organic material and have a moisture content of approximately 80 per cent and weigh 40-60 pounds per cubic foot.

Fine screens with openings of 1/16 to 1/8 inch are used to pretreat industrial wastes or to relieve the load on sedimentation basins. This type of screen will remove as much as 20 per cent of the suspended solids. Fine screens should be preceded by a coarse screen or a shredder to remove the larger particles.

Figure 1 shows representative volumes of screenings which can be expected for different spacing of bars. However, considerable variations from these figures occur and they cannot be explained by screen openings alone. Other factors affecting the quantity of screenings are the percentage of combined sewers in the tributary system, the character of industrial wastes treated and the habits of the tributary population. It has been observed that the sewage treatment plants serving penal institutions and mental hospitals have much greater quantities of rags in the screenings than is considered normal.

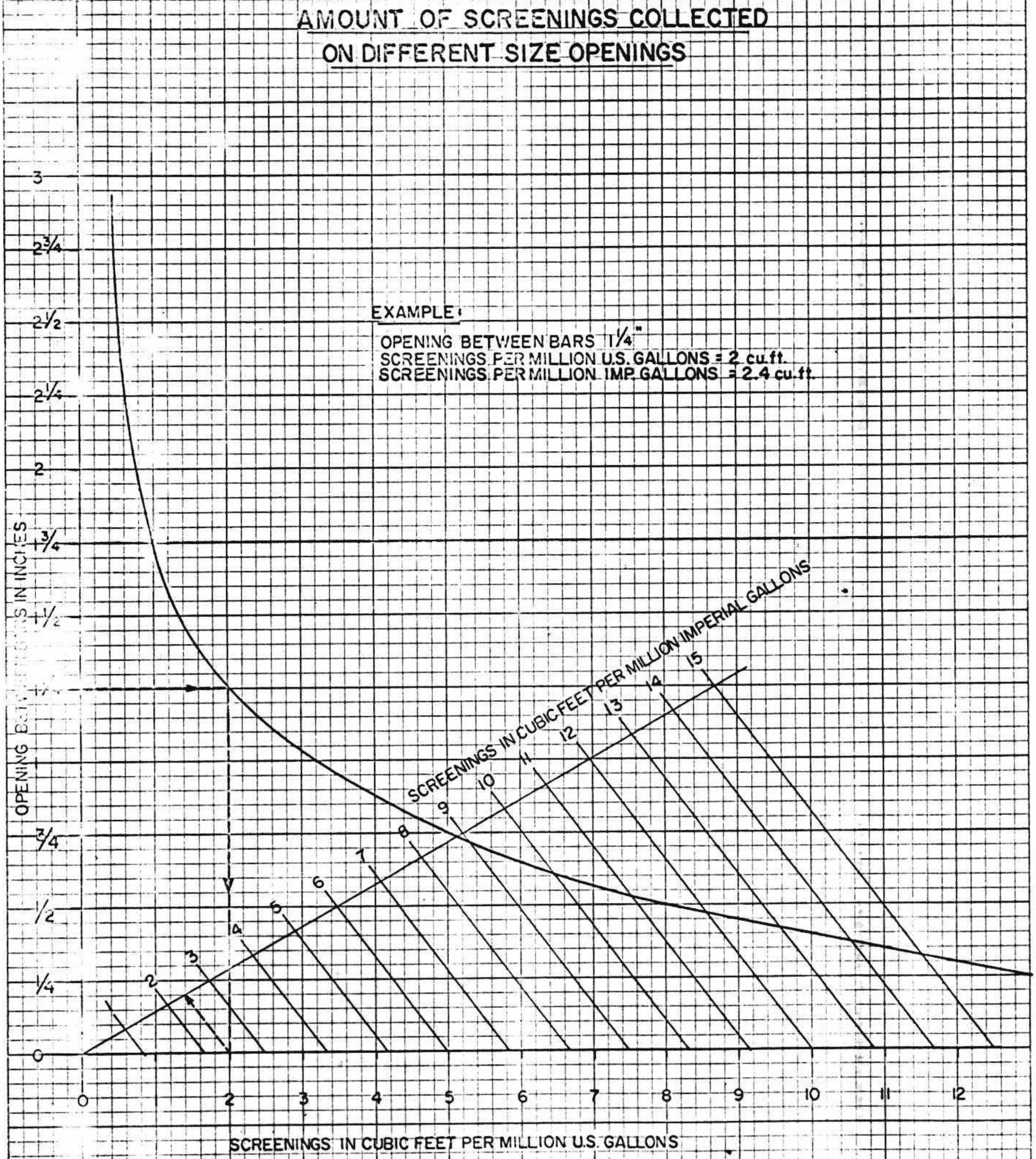


FIGURE I

AFTER: H.G. HODGSON

DISPOSAL OF SCREENINGS

The disposal of screenings is a matter that should receive serious consideration since, even if they are very thoroughly washed, they may become extremely obnoxious. Methods commonly employed include incineration, burial, drying with raw sludge, and maceration and return to the flow. The method used will generally depend on the size,type and location of the treatment plant.

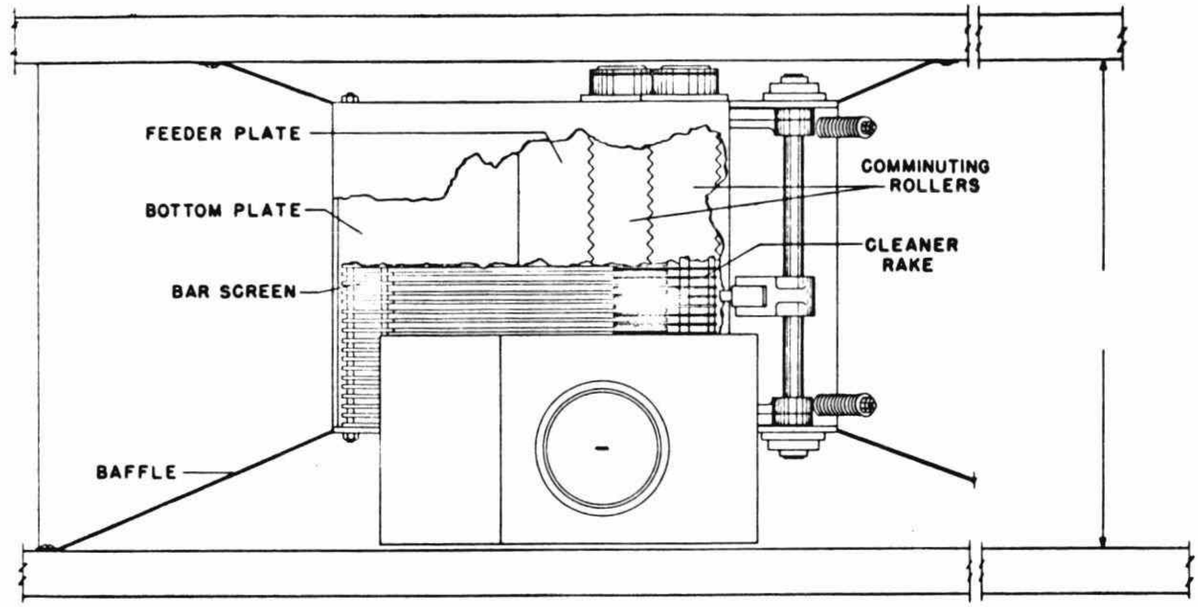
Burial is especially applicable for the medium screenings of small plants. They should be buried in trenches and immediately covered with a layer of earth or washed grit. If the screenings are allowed to remain on drying platforms or in drying baskets for removal only one or two times a day, offensive odours can be prevented by sprinkling them liberally with powdered lime. The screenings should be buried sufficiently deep to avoid odours and shallow enough to permit bacterial activity. A cover of 12-18 inches will probably give the best results.

In the larger plants employing medium screens, the screenings may be disposed of by burial at a municipal sanitary landfill site or at the municipal dump. If the screenings are to be removed from the plant for disposal, they should be deposited in covered cans either immediately after they are removed from the screens or after allowing a short period of dewatering. In the warmer months in order to keep odour and fly problems to a minimum, the screenings should be washed down by hosing them on the drying platform.

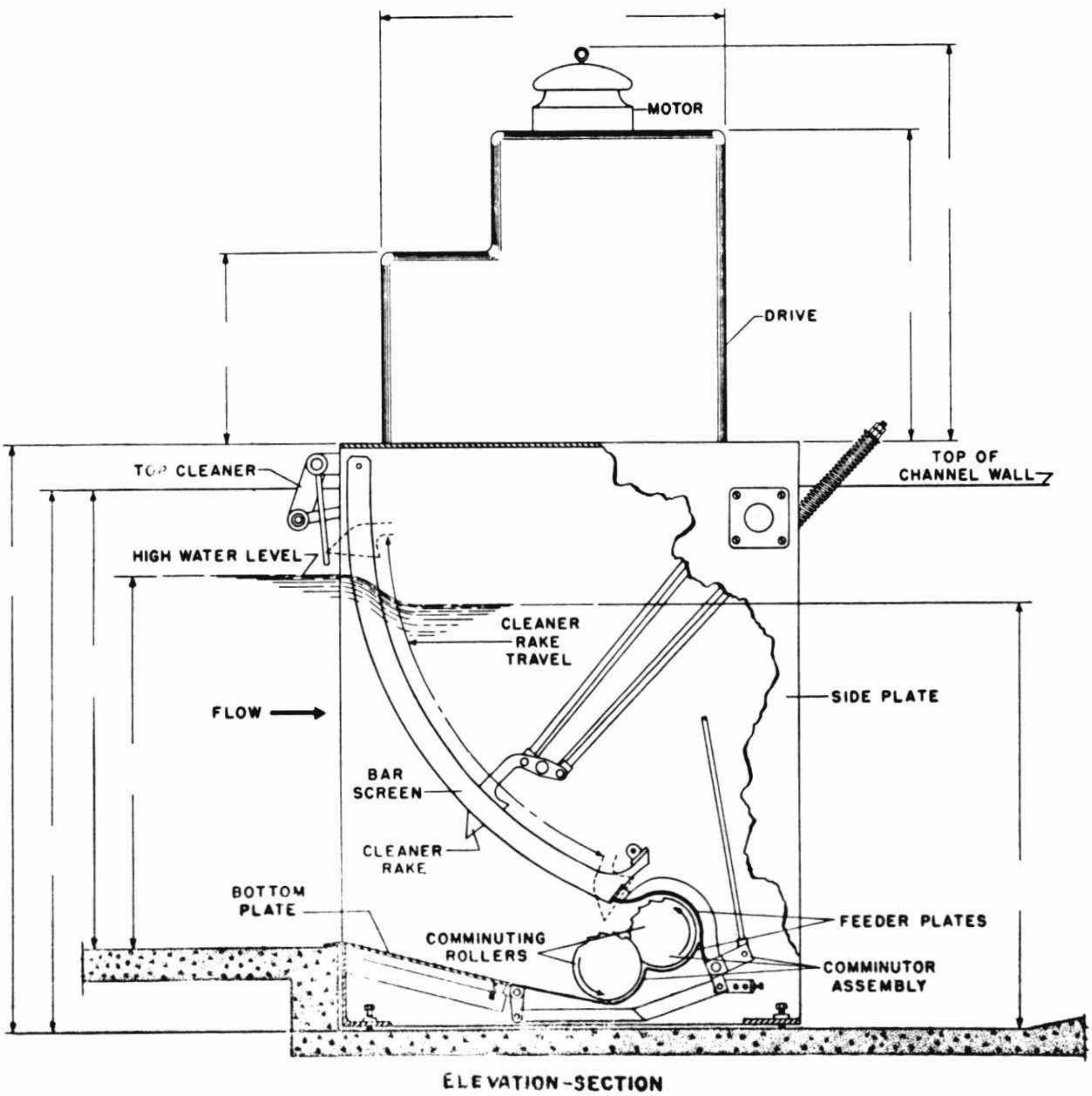
Incineration of screenings has been found to be moderately satisfactory at some plants in the United States. However, this method is not used in Ontario at the present time and most of those in the States have been abandoned, except at the largest plants.

Screening grinders are widely used for the disposal of screenings. The material is reduced in size and returned to the raw sewage or mixed with sludge depending on the location of the grinders in relation to other treatment processes.

Screenings from the grinder either settle out in the primary settling tanks or are mixed directly with the raw sludge. The screenings become part of the raw sludge or primary scum introduced to the digestion tank and are decomposed by the same action as the other constituents thereof. It is, therefore, desirable that the screenings be ground as fine as possible. Some digester foaming or excessive scum blanket formation has been attributed to this method of disposal of screenings but provision in the digestion tanks for better scum control can overcome this disadvantage.



PLAN



ELEVATION-SECTION

Figure 2

Comminuting devices combine the functions of a screen and a disintegrator and have a number of advantages. They save the necessity of removing the screenings from the flow, thereby eliminating the unsightliness and nuisance normally associated with this part of the plant. In large plants they are usually protected by a coarse bar screen and to facilitate maintenance they are installed either in duplicate or provided with a bypass fitted with a hand-cleaned screen.

The three major types of sewage and screening grinders are described briefly below because they may be of interest to the operator who may at some time have one installed or who may have the opportunity to operate a plant with such equipment.

One type includes a stationary bar screen, an oscillating rake, and two rotating cylinders, each having toothed segments. The screenings retained on the bar are moved downward by the rake into the space between the rotating cylinders located near the bottom of the channel. As the teeth of the two cylinders are moving in opposite directions at the same time, there is an intensive tearing and grinding action which reduces the screenings to particles small enough to pass through the bar screen.

The comminuting devices consist of a revolving-slotted drum screen. The screen is almost completely submerged in the flowing sewage. The sewage passes into the drum through slots and is discharged through an opening in the bottom. Any material too large to pass through the slots is cut by cutters on the drum into pieces small enough to pass through. A third cutting member, called a shear bar, cuts any stringy material that may pass only part of the way through the slots.

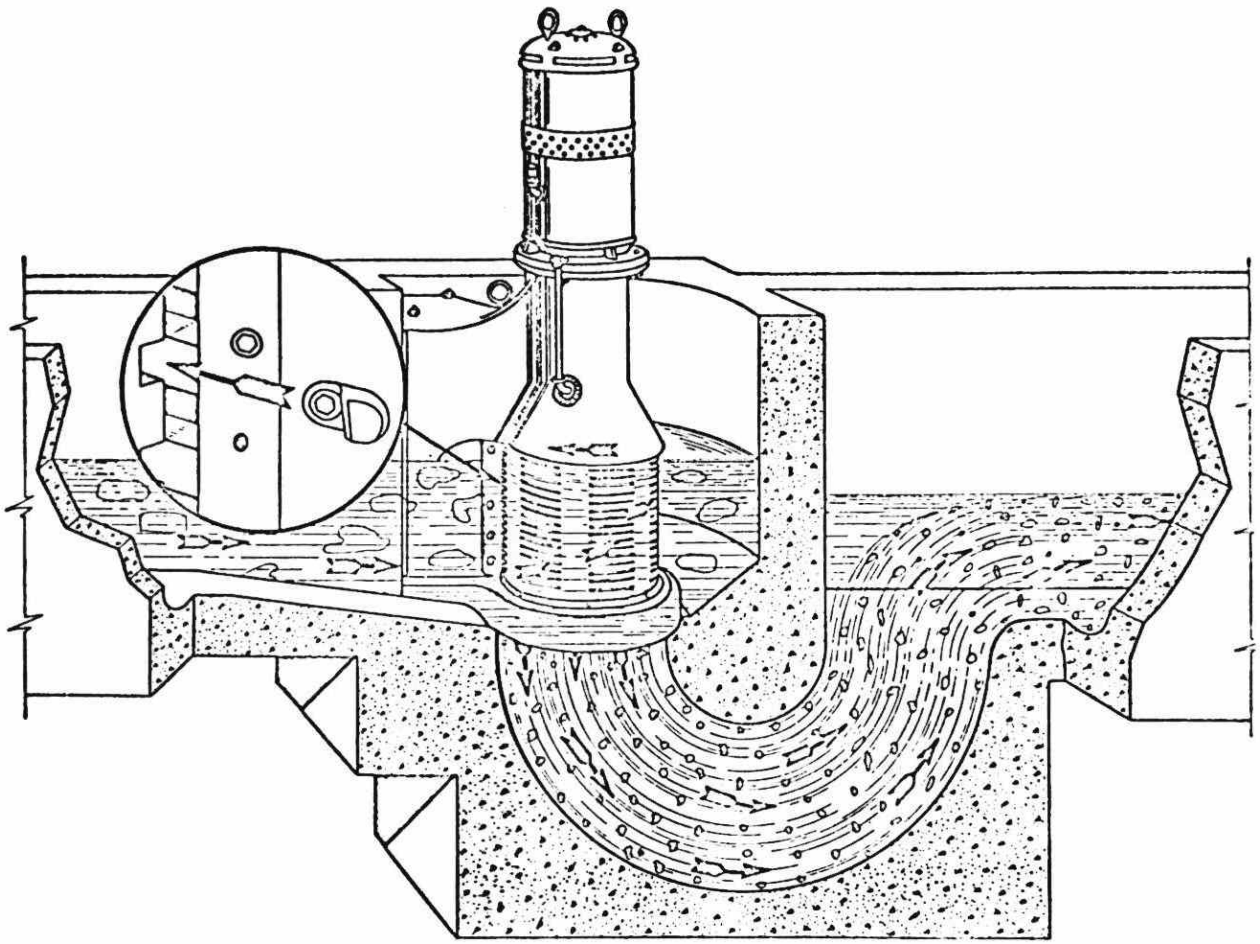
The third general type of screenings shredder is located aside from the bar screen. The screenings are usually removed from the screen mechanically and then discharged to the disintegrator. The shredding device may be constructed similar to a household meat grinder (shear action), or it may be of the swing hammer type. A sorting tray may be provided so that larger harmful items (shoes, metal, rocks, bottles) may be removed.

GRIT REMOVAL

There are three major categories into which removal facilities may be placed.

i.e.

1 - grit channels
2 - aerated grit removal chambers
3 - mechanical devices



CROSS-SECTION OF COMMINUTOR

FIGURE 3

QUANTITY OF GRIT

As in the case of screenings, it is not possible to state how much grit can be expected at each plant. In the basic course we discussed the many variables affecting the volume of grit. Average figures for the expected quantities of grit are 8 cubic feet per million gallons for combined sewer systems and 3.5 cubic feet per million gallons in separate sewer systems.

RECTANGULAR GRIT CHANNELS

The rectangular grit channel equipped with a proportional weir at the outlet end is the most common type of grit removal facility presently employed. Figure 4 shows this unit. The proportional weir is used to automatically control the velocity of flow. The units are designed so that the velocity of flow is maintained at as close to one foot per second as possible, although the depth of flow may vary. As noted on the diagram the line of settling of the 65-mesh grit terminates at the bottom of the channel. (A 65-mesh grit indicates that the grit is such that it will pass through a screen having 65 openings per linear inch or 4225 openings per square inch). It should be noted that temperature does effect the rate of settling.

The removal of the accumulated grit in this type of unit varies by degrees from a completely manual operation to a completely automatic system.

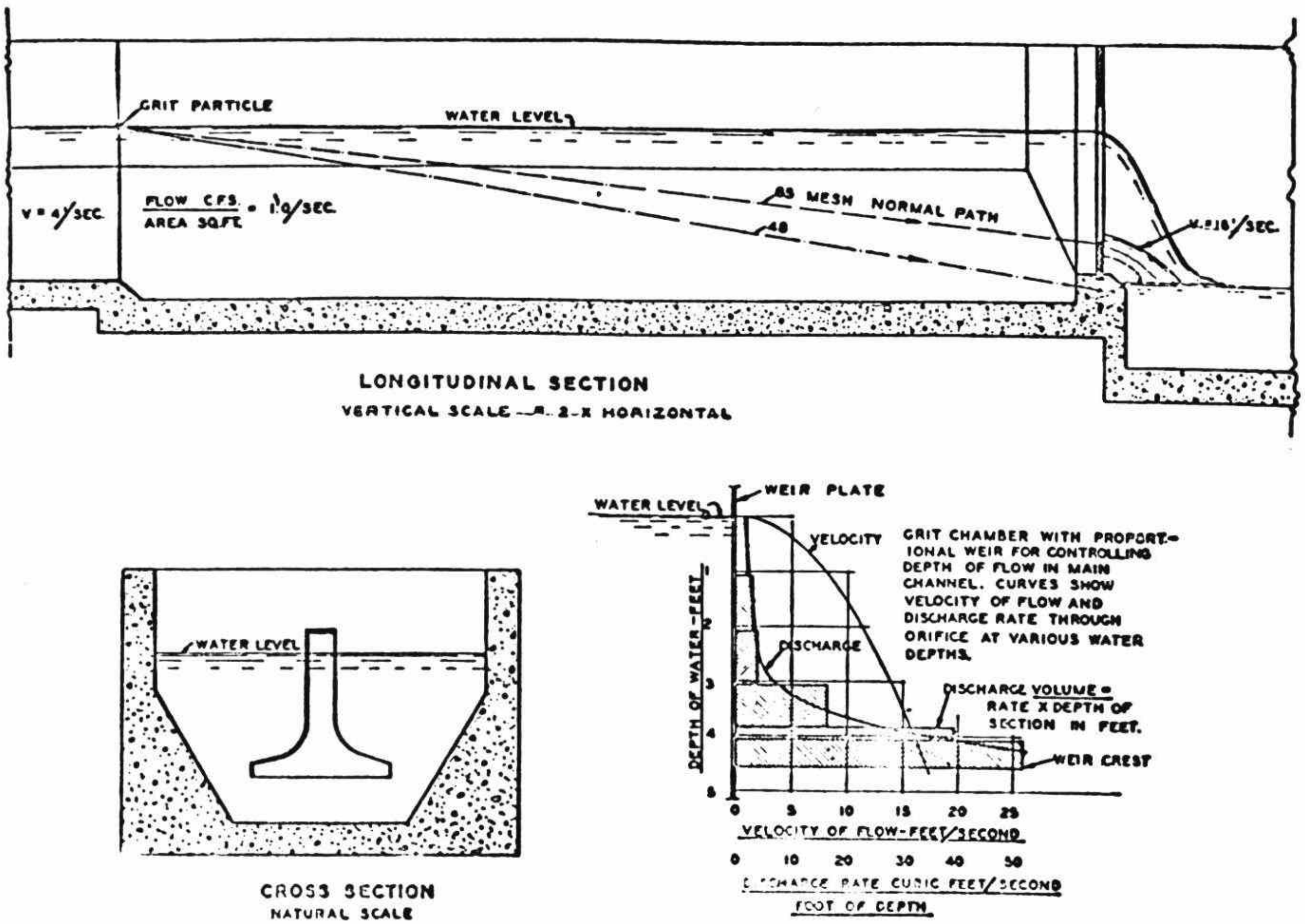
The simplest method of grit removal, by shovel and wheel barrow, is used in the small and medium sized plants. In plants where only one mechanically cleaned channel is provided, a manually cleaned unit may be employed as a bypass. Automatic or semi-automatic collectors are usually conventional conveyor-type equipment with buckets, plows, scrapers, or rakes. A grit washing or classification device may be included, such units will be briefly discussed at a later point.

AERATED GRIT REMOVAL CHAMBERS

Grit can be made to settle in a spiral flow aeration tank and will accumulate beneath elevated air diffusers mounted along one of the longitudinal walls of the tank.

In application the aerated grit chamber is similar to a standard spiral flow aeration tank with diffused air tubes located on one side approximately 2 feet above the bottom of the tank. Hoppers are provided beneath the diffusers for collecting the grit. Flow enters the tank at the opposite end of the effluent weir and in a direction to coincide with the roll of the tank contents.

When the sewage flows into the tank the grit particles will tend to settle to the bottom at rates effected by the particle



RECTANGULAR GRIT CHANNEL FIGURE 4

size and the roll of the tank, the roll being controlled by the rate of air diffusion and the shape of the tank. The heavier particles with their correspondingly higher velocity of settling drop to the bottom, whereas the lighter organic particles are carried with the roll and eventually discharged from the tank.

Studying the action of a single particle of grit, it will be seen that as it follows the roll along the bottom of the tank it will have a tendency to leave the flow and settle to the bottom. If the particle starts through the tank on the outside of the spiral, it will settle out on the first roll; whereas, if it starts toward the inside of the spiral, it may require three or four rolls to reach the outside, make contact with the tank bottom, and settle out. If no short-circuiting occurs, the only particles that do not settle must be such that the velocity of the roll will retain them in suspension and carry them out with the effluent.

The continuous roll of the tank contents promotes an equal detention for all portions of the flow.

The removal of particles of grit 0.2 mm. and greater will protect pumps from heavy wear. Particles of this size are carried with the roll to the tank bottom, where the velocity is great enough to wash them towards the grit hopper. Because there is insufficient velocity to lift the grit particles they settle along the edge of the tank under the diffusers, where they are collected in hoppers.

Flow velocities of 2 feet per second are developed in aeration tanks with air supplied at the rate of 3 cfm. per foot of tank length. A velocity of 0.75 fps. is required to move sand (0.2 mm.) along the tank bottom whereas a velocity of 6 fps. is necessary to start an upward vertical movement. The lighter organic particles are held in suspension by the velocity of 2 fps. In addition, the scouring action resulting from the grit rolling across the bottom, loosens any organic matter adhering to it.

Variations in flow through the tank seem of little importance.

Removal of the accumulated grit from the hopper may be effected by:

1. Clamshell bucket
2. Air lift (if lift is not greater than 5-10' above liquid surface)
3. Jet pumps

One disadvantage to aerated grit removal is that it produces a clean grit. As grit yield goes this may be all right but many operators also want to remove much of the cellulose and hair-like particles that if not taken out with the grit, add to

the digester inert scum layer. A detention type tank is preferred in these cases.

DETRITUS TANKS

Short-period sedimentation in a tank that operates at substantially constant levels produces a mixture of grit and organic solids called detritous. The lighter organic solids are subsequently removed from or washed out of the mixture.

Several manufacturers specializing in sewage disposal equipment have perfected this type of equipment. The Dorr Company has developed one such unit called the ''Detritor'' and this device is shown in figure 5. It is noted that the term detritor and the function thereof is not only to remove the grit but also to wash the grit. A subsequent section of this lecture will deal with washing or classification devices, at this point we will only deal with the detritus tank.

The grit collecting mechanism is installed in a square, shallow, concrete tank with filled in sloping corners. Sewage enters along one side of the tank through adjustable vertical gates which are set to provide a uniform influent velocity across the entire width of the unit. After entering the sewage flows in straight lines across the tank and overflows at a weir constructed along the outlet side of the tank.

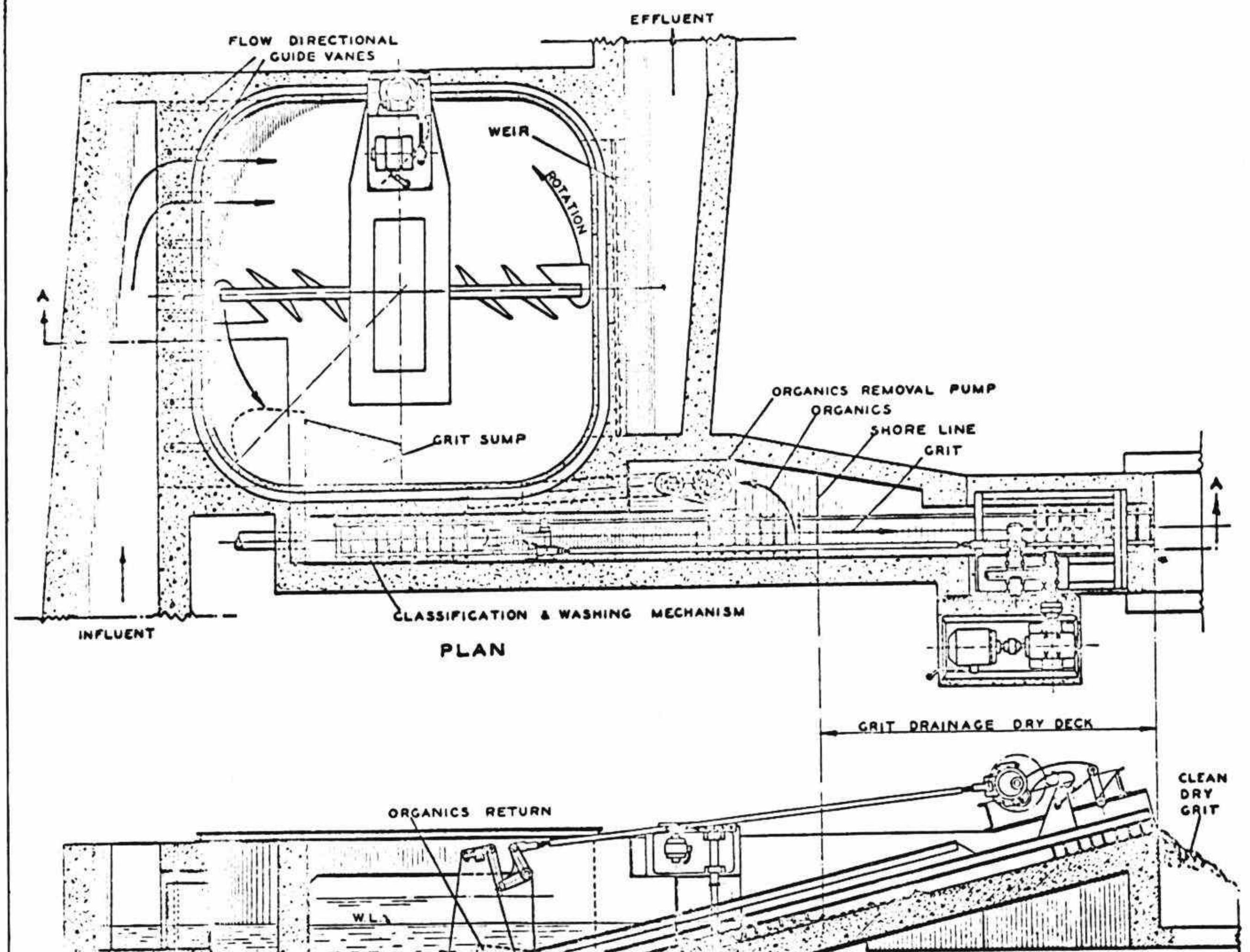
The collecting mechanism consists of two structural steel arms, attached to a vertical shaft and fitted with outward raking blades with scoops on the ends. As the rakes revolve, settled grit is plowed outward to the radius where the end scoops collect and discharge it to a hopper at one side of the tank.

The grit is then discharged to a washing or classifying device. Such a unit will be discussed later.

CLEANING GRIT

In order to obtain a relatively clean grit (less than 3% volatile matter) various methods of washing out the excess organic materials have been developed.

1. Elevation on a sloping ramp by means of a series of reciprocating rakes (refer to figure 5). The reciprocating rakes convey the grit up the inclined tank and at the same time impart a rolling action which releases the entrained organic matter. The organics are pumped back to the influent end of the tank by a pump which acts similarly to a propellor in a draft tube. Excess moisture drains off as the grit progresses up the latter part of the ramp.
2. Elevation by screw conveyor in a sloping trough. The action of this type of grit washer is basically the same as that described for the reciprocating rake process.



DORR DETRITOR

FIGURE 5

3. Aerated grit chambers -- the use of diffused air for grit removal as previously described produces a well "washed" or classified grit.

4. "Cyclone" classification units -- a diagram of one type (Dorr) of these units is shown in figure 6. The slurry enters the cylindrical feed chamber tangentially and develops a cyclonic vortex pattern. Centrifugal forces throw the grit contained in the slurry to the walls of the cone. As these solids collect along the walls of the cone, they move towards the apex and discharge through the apex valve. The lighter grit-free liquid moves to the inner spiral of the vortex where it is displaced into the overflow opening (vortex finder). Usually the flow from this type of unit is wetter than that of the reciprocating rake type unit, however they remove grit up to a 150 mesh.

Under certain circumstances these units are used for the removal of grit from raw sewage.

DISPOSAL OF GRIT

Clean grit is characterized by the lack of odours which would normally result from the presence of decomposing organic matter. Except when grit is carefully washed, it will contain up to 50 per cent organic matter (mostly garbage) by weight. It is inevitable that some food value will be found in such grit and consequently it will become an attraction to insects and rodents as well as unsightly and odorous.

In the majority of cases, grit is disposed of by burial or at the municipal dump or landfill site. At the plant, unwashed grit which is removed from the removal facility should be kept in cans and should be removed to the disposal site at least daily. If the grit is adequately washed (less than 3% volatile) it may be used as fill around the plant or it may be used to re-sand sludge drying beds.

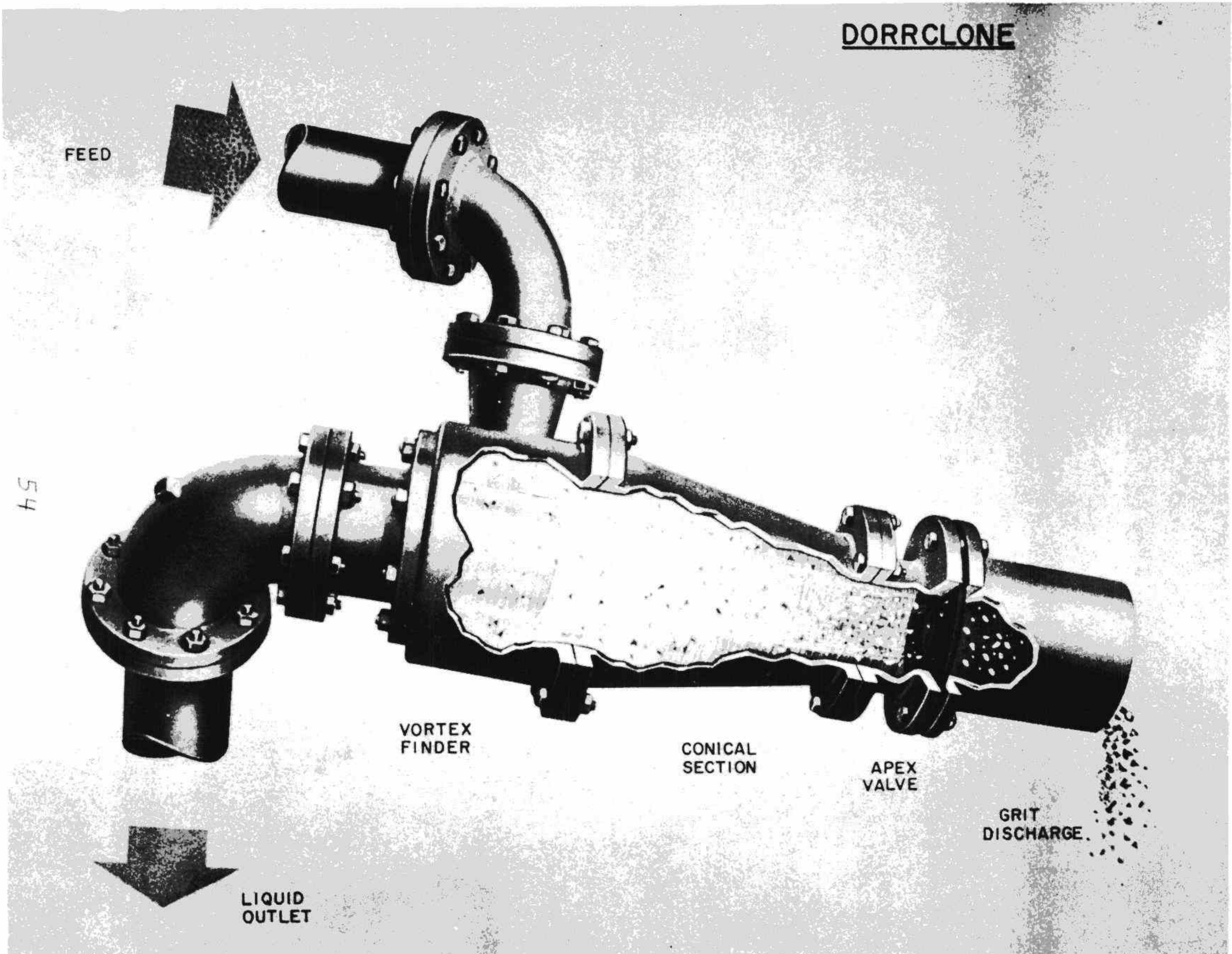

DORRCLONE
FEED
VORTEX
FINDER
CONICAL
SECTION
APEX
VALVE
GRIT
DISCHARGE
LIQUID
OUTLET

FIGURE Nº.

THE SIGNIFICANCE OF SOME OF THE ANALYSES USED IN SEWAGE PLANT CONTROL

by

A. J. HARRIS

Assistant Director of Laboratories

An Address To
The Ontario Water Resources Commission
Intermediate Sewage Works Operators' Course
Toronto, Ontario
March 5, 1962

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INTRODUCTION

The complete analyses of sewage or sewage effluent is a difficult matter unless the probable nature is known beforehand. Analytical results are expensive to produce and it is necessary for economy to do only those considered adequate to judge the efficiency of the operation. The limiting analyses with which an operator may judge his effluent quality is usually the 5-day biochemical oxygen demand and suspended solids. Other aids are pH, the four nitrogens, volatile acid and the coliform bacteria count.

To have control over a treatment plant there must be a running record of the chemical analyses. All that was originally known about activated sludge was that if you put enough air into the mixed liquor and kept it there long enough, the process performed satisfactorily. Naturally such type of operation does not give the operator any tools to anticipate trouble. These tools or analyses will now be discussed in the order of their relative importance.

Five Day Biochemical Oxygen Demand

The 5-day biochemical oxygen demand test (BOD) is used as a standard method of measuring the oxygen demand of sewage. The fact that sewage, sewage effluent, or other material has little or no BOD does not mean that it is harmless. The BOD indicates how much oxygen must be available to provide natural purification by oxidation or the amount of oxygen utilization due to aerobic action of bacteria and protozoa feeding upon the organic constituents of waste water. It is the most important single characteristic, from the stand-point of waste water treatment and water pollution control, in measuring SDP performance.

In a sewage plant, all organic matter that can exert an immediate BOD is decomposed, the long term putrescible organic matter leaves in the effluent to be decomposed biologically in the receiving water. The oxidation of sewage takes place in two

stages. In the first stage it is mainly the carbonaceous matter which is oxidized to carbon dioxide and water, and in the second stage, the nitrogenous matter with the formation of nitrates. The first stage completes in about 20 days or is two-thirds complete in 5 days. The second stage, or nitrification, usually does not set in until after 7 days. Thus the reason for choosing the 5-day figure for BOD test is that most of the carbonaceous matter (about two-thirds) will be oxidized while nitrification which may continue for many weeks, has not yet had a chance to interfere with the results.

All sanitary chemical analyses are usually expressed in parts per million and the 5-day BOD for domestic sewage is usually greater than 100 ppm and less than 300 ppm. Sewage from different towns show marked variation in the amount of organic matter present. The presence of certain types of industrial wastes, such as from abattoirs, canneries and tanneries, will of course increase the concentration. Values of 500 ppm or more are not uncommon for raw sewage.

The ease or difficulty with which a sewage can be treated by biological oxidation method and therefore the capacity of the SDP required are not necessarily related to the strength of the sewage, but depend upon the nature of the substances present and their toxicity to micro-organisms responsible for the purification. This means that a sewage of 1,000 ppm BOD may not be as difficult to treat as another of 100 ppm BOD.

In order for you to realize the value of a BOD between 100 ppm and 300 ppm normally found in sewage, I have obtained BOD figures for various substances.
They are in parts per million,

blood	- 165,000
whole milk	- 100,000
skim milk	- 75,000
beet sugar wastes	- 50,000
paper mill wastes	- 25,000
brewery wastes	- 10,000
cannery wastes	- 7,500
alcohol	- 1,000
starch	- 1,000
glycerol	- 500
sugar	- 250
tap water	- 0.5

Also, the classification of a stream according to the BOD would be as follows:

1 or less	- very clean
2	- clean
3	- fairly clean
5	- doubtful
7.5	- poor
10	- bad
over 20	- very bad

Sewage arriving at a plant may be fresh, that is, with some dissolved oxygen present or stale, that is, with the dissolved oxygen exhausted. It may be septic owing to the development of anaeorbic organisms which reduce sulphates and organic sulphur compounds to hydrogen sulphide. Septic sewage is more difficult to treat than fresh or stale sewage. Pre-aeration makes stale or septic sewage much easier to treat.

The BOD of a primary effluent will indicate how much is removed by settling alone. It is normal for the BOD to drop by 30 percent or more during primary settling giving effluents between 65 ppm and 200 ppm BOD. Of course, the effects of digestor supernatant if bled to the primary should not be disregarded. Three thousand gallons of digestor supernatant in a primary tank before sampling could give very discouraging results. Because digestor supernatant is anaerobic, ie. septic, the BOD is misleading. Supernatant may show a figure of 200 ppm BOD, the actual BOD is not apparent until the facultative organisms become aerobic. Samples from the primary effluent should be collected before supernatant from a digestor is added.

The BOD of a mixed liquor in the aeration chamber will be at a maximum when the sewage is introduced and from that point on the rate will decrease, first rapidly, then more slowly. Reference to the BOD record will indicate that if extra strong sewage is the cause of rising sludge, then sludge concentration should be increased. If no unusually high BOD has been recorded then the dissolved oxygen is probably low and the rate of aeration, especially at the head end of the tanks, should be increased.

The BOD of a sewage effluent discharging to a stream where it receives at least a ten-fold dilution by clean water, should not exceed the standard of 20 ppm. An effluent with low suspended solids and a BOD below 20 ppm will be sparkling clear, have no odour and will be rapidly assimilated in the receiving water.

The chief limiting factor in the purification of sewage effluents by dilution is that fully aerated river water contains only comparatively small amounts of oxygen - 8 to 10 ppm. Hence, the complete oxidation of sewage in a river might easily require a distance of 10 to 30 miles travel in a stream.

Sometimes, although the BOD indicates a quality effluent of 20 ppm, stream appearance may not be good. The reason for this is that an effluent of 20 ppm BOD saturated with oxygen has a similar demand on the oxygen of a stream as one of BOD 11 ppm. denuded of oxygen, and the immediate effect of the lower BOD effluent might well be more harmful. Particularly is this the case when inadequate dilution of the effluent is present.

It can be said however, that the BOD estimation still remains the best available single test for assessing organic pollution but its errors, shortcomings, and limitations must be

borne in mind and the results of any test correlated with other analytical data.

SOLIDS

Solids are determined by evaporating the sample at 103^{0}C. (217^{0}F or 5 degrees above boiling) and weighing the residue. Suspended solids are the most significant in indicating the efficiency of settling in the day to day operation. Usually suspended solids of sewage or an effluent will run at about the same concentration as, or slightly higher than, the BOD. For example, a raw sewage of 200 ppm BOD will likely have a suspended solids of around 200 ppm and an effluent of 20 ppm BOD will have a suspended solids of about 20 ppm.

Primary settling will reduce the suspended solids by 40% to 60% and the removal is quite often better than the percent BOD removal.

The mixed liquor suspended solids (MLSS) will be of your choosing and will depend upon the strength of raw sewage, tendency to foam, freshness of return sludge and such factors. Usually 1,000 ppm to 3,000 ppm MLSS are required or about 10% to 20% by volume as measured for one hour in a graduate.

The suspended solids in a good effluent, if any, should have a white ashy appearance, be fluffy and not exceed 20 ppm if discharged to a swift flowing stream.

The percent removal of suspended solids is frequently an indication of the adequacy of a plant. If the plant cannot reduce suspended solids, it probably cannot remove BOD effectively. Secondary treatment generally removes 90% to 95% of the suspended solids leaving a clear, water like effluent. In comparing effluent quality to raw sewage, the time lag should be considered or allowed for in sampling and also the hourly fluctuation in the composition of the sewage entering the plant.

Some actual figures for suspended solids at locations in 3 sewage plants are as follows. For comparative purposes, the BOD is shown in brackets.

	1		2		3	
	S.S.	(BOD)	S.S.	(BOD)	S.S.	(BOD)
Raw	186	(240)	387	(360)	235	(300)
Settled	85	(210)	187	(236)	109	(168)
Final	22	(19)	11	(13)	7	(8)

The <u>dissolved</u> solids figure on an analytical result sheet gives little operating information to an operator. Values for dissolved solids will represent, in many cases, the dissolved solids such as hardness, chlorides, etc. present in the town water supply.

An analytical result sheet usually shows three columns for solids. These are total, suspended and dissolved solids. The centre column, or suspended solids, carries the significant figure for sewage plant operators.

pH

pH has no sanitary significance but is used to control the operation of a sewage plant. The term pH is an abbreviation which is used in chemistry to represent a scale of numbers from zero to fourteen and shows the intensity but not the amount of acidity or alkalinity. Seven is the neutral point, below 7 the acidity becomes more intense and above 7 the alkalinity becomes more intense.

Normal raw sewage will arrive at a plant with a pH between 8 and 9 or slightly alkaline. This is desirable in biological oxidation since it is the range that the micro-organisms feeding upon the sewage prefer. Sewage arriving at the plant below 6 or above 9 can be suspected of containing an industrial waste.

During oxidation processes of sewage treatment there is a decrease in alkalinity caused by oxidation of carbonaceious matter to acidic substances, assimilation of ammonia by micro-organisms, and oxidation of ammonia to nitric acid. pH determinations therefore give some indication of the degree of oxidation at any stage during the treatment of sewage.

The pH test of sludge will aid in determining whether it is well digested. Well digested sludge will have a pH between 6.6 and 7.6 while badly digested sludge is usually below 6.0. The addition of lime to a digester should cease when the pH reaches 6.7.

NITROGENS

The nitrogens include organic and ammonia nitrogen and nitrite and nitrate. Sewage contains a large amount of organic matter in the form of nitrogen, mainly as ammonia and organic nitrogen. In the stages of purification at a sewage plant by means of dissolved oxygen in the aeration chamber or trickling filter, ammonia and organic nitrogen are converted to nitrite (NO_2) and by taking up additional oxygen, to nitrate (NO_3). The nitrate (NO_3) is stable and when discharged in the effluent is used up as an aquatic plant food and disappears from the stream.

(a) Ammonia Nitrogen (NH_3)

Fresh sewage is usually low in ammonia nitrogen but if it becomes stale and bacterial decomposition sets in, then ammonia will increase at the expense of the organic nitrogen. Raw sewage will contain from 15 to 50 ppm of ammonia with little change occurring in the primary tank. The effluent from activated sludge or a trickling filter should show no ammonia. Poorly treated sewage will contain ammonia and the presence of ammonia in an effluent affords a specially delicate chemical test for detecting sewage

pollution in a stream.

(b) Organic Nitrogen (Kjeldahl)

The name Kjeldahl refers to the name of the man who developed the test for organic nitrogen. It may be carried out to include or exclude the ammonia. Since the two forms of nitrogen are inter-related, that is, if one decreases, the other increases or vice versa, this laboratory uses the method which includes the ammonia with the organic nitrogen. This eliminates changes that may occur between the time of sampling and the time of analyses since the sum of the two will remain the same.

Values for organic nitrogen in raw sewage will vary from 25 to 85 ppm. and effluents from biological oxidation processes from 5 to 20 ppm. Of course, the presence of large quantities of nitrogenous industrial wastes, or large volumes of nitrogen deficient wastes may change these figures considerably.

The proper amount of organic nitrogen is essential for the successful operation of biological oxidation processes. Nitrogen is one of the most important nutrients or food materials which satisfy the requirements of the micro-organisms. A ratio of one part of nitrogen for every 20 parts of BOD is absolutely essential. Fortunately, raw sewage usually contains a C:N ratio of about 3:1 or six to seven times the required amount.

(c) Nitrites (NO_2) and Nitrates (NO_3)

Since nitrites are only an unstable transient stage between organic nitrogen and nitrate, nitrite and nitrate may be discussed together. In biological treatment processes, organic and ammonia nitrogen are first converted to nitrite by oxygen and by taking up additional oxygen converted to nitrate. Thus well stabilized effluents can be expected to contain a quantity of nitrate and some nitrite which has been a little slow in converting to nitrate. In fact, a good effluent should contain not less than 10 ppm of nitrate. The raw sewage will contain less than 0.5 ppm of nitrate and even less of nitrite.

To summarise the nitrogen analyses, a typical raw sewage and final effluent from a biological oxidation plant would be as follows:

	ppm NITROGENS			
	Ammonia (NH_3)	Organic (Kjeldahl)	Nitrite (NO_2)	Nitrate (NO_3)
Raw	35	60	0.06	0.3
Primary	35	60	0.06	0.3
Final	0	7.0	0.9	8.0

As a matter of interest, the quantity of nitrogen in sludges, on a dry bases, are about 2% for primary sludge; 5 to 7% for activated sludge; and from 2.5 to 4% for digested sludge.

VOLATILE ACIDS

When sludge is stored for any length of time, it undergoes fermentation by anaerobic bacteria that thrive under acid conditions. These bacteria, however are replaced by others which induce an alkaline fermentation and eventually the sludge undergoes fairly complete digestion with production of gas.

The determination of volatile acids is valuable primarily as an index of the progress of anaerobic biological digestion of organic matter. In the production of gas during the decomposition of organic solids in a digester there is first a combination with water to form the simpler organic acids such as acetic, these acids then decompose to give a gas containing about 70% methane and about 30% carbon dioxide. The limit of volatile acid for smooth continuous fermentation has been found for most materials to be about 2,000 ppm calculated as acidity. Values below 2,000 ppm seemingly have no undesirable effect on digestion.

BACTERIOLOGICAL EXAMINATION

The bacterial analyses carried out on sewage and sewage effluents are for the coliform group of bacteria. In other words, the examination is for intestinal bacteria normally found in domestic sewage. The results are expressed in the "most probable number per 100 millilitres" (MPN per 100 ml.).

The bacteria in sewage may be harmful or harmless. The harmful ones are called pathogens and include those causing typhoid, paratyphoid, dysentry, cholera and gastro-enteritus,

The numbers of bacteria in sewage has little or no significance in the interpretation of analyses. Normally 10 to 200 million per 100 millilitres are found in fresh sewage but this can increase, upon standing in a sample bottle, to 1,000 million if unrefrigerated. In sewage treatment, some settle out in the sludge and unfavourable environmental conditions through the application of air and the drop in temperature across the plant will reduce the number of bacteria. Those surviving in the effluent may begin to multiply again if the conditions in the receiving water are favourable.

The allowable limit in waters used for recreational purposes is 2,400 MPN per 100 ml. and for a municipal supply it is about 2 MPN per 100 ml. A sewage effluent, after complete treatment, can contain up to one million coliform bacteria per 100 ml. The chemical quality, ie. BOD etc., of an effluent is no guide as to the number of faecal organisms it may contain. Thus chlorination of an effluent is essential if the receiving water is to be utilized for public consumption or for recreational activities. It is inferred that if organisms of the coliform group, which can easily be detected

and counted, cannot be found, then the pathogens must also be absent.

A normal reduction of coliforms in an activated sludge plant is from 80% to 95%. This reduction will increase to 99.99% with adequate chlorination of the effluent. It is usually not possible to completely sterilize an effluent with the application of chlorine because some bacteria or their spores are protected by a coating of organic matter present as suspended solids and are not penetrated by the chlorine.

The following are round average figures for bacteria in sewage treatment:

Stage	Number per 100 ml.
Raw sewage	100 million or more
Settled sewage	2.5 million or more
Activated sludge effluent	0.1 million or more
Chlorinated effluent	1 thousand or more

DETERMINATION OF SLUDGE REQUIREMENTS IN THE CONVENTIONAL ACTIVATED SLUDGE PROCESS

by

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An Address To
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March 6, 1962

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INTRODUCTION

The activated sludge process is utilized to convert non-settleable substances in the sewage in finely divided, colloidal and dissolved form into settleable sludge. This newly formed sludge is removed in final settling tanks and is either returned to the aeration tanks or wasted to the primary settling tanks. The raw sludge and waste sludge formed in the primary settling tanks are removed as mixed sludge to the sludge treatment facilities.

Activated sludge is a continuously changing medium in which different groups of micro-organisms primarily bacteria and protozoa in the presence of dissolved oxygen are increasing and decreasing in activity due to varying nutrients in the sewage and to varying environmental conditions. By the action of their growth these organisms convert the pollutants absorbed on the sludge and dissolved in the sewage to carbon dioxide, sulphates, nitrates and the living protoplasm of their own bodies.

From this brief description it should be realized that the activated sludge process is one which does not readily lend itself to simple analyses and mathematical formulae.

It is hoped that the following discussion of sludge requirements will assist advanced operators in the better operation of conventional activated sludge treatment plants.

AERATION TANK SLUDGE QUANTITIES

Recently it has been realized that the efficiency of BOD removal is related to the organic loading and the weight of solids carried in the aeration tanks.

Removals in activated sludge units average about 90% at BOD loadings up to 30 pounds per 100 of suspended mixed liquor solids. At loadings between 30 and 50 pounds, efficiency becomes somewhat erratic. It is suggested that for moderate to large plants, loadings between 30 to 40 pounds can be used successfully. Small plants which do not receive 24-hour attendance should use lower loadings of only 20 to 30 pounds of BOD applied per 100 pounds of aerator suspended solids. Lower loading ratios increase the air requirements, and shift the process into the total oxidation modification operating range.

With the size of the aeration tanks fixed and the BOD loading known the amount of suspended solids required in the aeration tanks can be calculated.

For Example

Sewage Flow = 1.0 MGD
5-Day BOD in Raw Sewage = 200 PPM
Primary Settling Tank BOD Removal Efficiency = 35%
5-Day BOD in Primary Effluent = 200 x 0.65 = 130 PPM
5-Day BOD Loading to Aeration Tanks

$$= \frac{130}{10^6} \times 1{,}000{,}000 \text{ (GALS.)} \times 10 \text{ LBS./GALS.} = 1{,}300 \text{ LBS.}$$

Assume Aeration Tank Volume = 40,000 CU. FT.

$$= 40{,}000 \text{ (CU. FT.)} \times \frac{6.25 \text{ (GALS.)}}{\text{(CU.FT.)}}$$

$$= 250{,}000 \text{ GALS.}$$

Assume Desired 5-Day BOD Loading - 20 LBS./100 LBS. of Mixed Liquor Suspended Solids

Total Pounds of Mixed Liquor Solids Required

$$= \frac{100 \text{ (LBS.)}}{20 \text{ (LBS.)}} \times 1{,}300 \text{ (LBS.)} - 6{,}500 \text{ LBS.}$$

Mixed Liquor Suspended Solids Concentration Required

$$= \frac{6{,}500 \text{ (LBS.)} \times 10^6}{250{,}000 \text{ (GALS.)} \times 10 \frac{\text{(LBS.)}}{\text{(GAL.)}}} = 2{,}600 \text{ PPM}$$

If the raw sewage strength increases to 250 PPM 5-Day BOD and the other factors remain the same, the mixed liquor suspended solids concentration should be increased to 3,250 PPM to maintain a steady loading ratio of 20 pounds of 5-Day BOD per 100 pounds of mixed liquor suspended solids.

It is still common practice to express the BOD loading in terms of the unit aeration capacity. Experience has demonstrated that BOD loadings of 25 to 30 pounds per day per 1000 cubic feet aerator capacity are the upper limit that can be handled in conventional plants.

In diffused air aeration tanks the suspended solids concentration is usually maintained between 1500 and 3000 PPM. In mechanical aeration plants the concentration varies between 500 and 1200 PPM.

With aerator solids held at 2000 PPM, a detention time of 5 hours, a settled sewage having a 5-Day BOD of 100 PPM, the BOD loading on the aeration tank is 30 pounds per 1000 cubic feet and 24 pounds per 100 pounds of suspended solids. Both of these values are within the limits normally accepted as good practice.

If the sewage flow is considerably below design or the sewage strength is greater than design it becomes obvious that detention time or aerator solids must be changed to compensate.

The process should be maintained at the 5-Day BOD loading to solids ratio that gives the best treatment efficiency. The optimum quantity of activated sludge carried is therefore expected to vary from plant to plant.

Although the concept of BOD loading in terms of activated sludge solids carried is an extremely important one, its application in practice must be seasoned with a knowledge of air requirements and the ability of the final clarifiers and the return sludge system to handle the solids involved.

RETURN SECONDARY SLUDGE QUANTITIES

The inter-relationships of return sludge flow; return sludge settled volume and return sludge suspended solids with sewage flow; mixed liquor solids; mixed liquor settled volume; and Sludge Volume Index were discussed in considerable detail during the first Sewage Operators Course held in 1961.

The various formulae available for determining return sludge quantities are summarized in Exhibit 1.

Ideal values for the variables of the conventional activated sludge process required by the formulae were assumed to give the percent return sludge figures shown in the summary.

There is a close correlation among the formulae although different tests are required for each.

Two formulae apply where only the suspended solids test is conducted. Two other formulae may be computed where only the 30-minute settling test is carried out. Where both of these control tests are made all five formulae may be used by the operator to establish the best return sludge rate and solids concentration for his plant.

WASTE SECONDARY SLUDGE QUANTITIES

The weight of excess secondary sludge to be wasted may be dependent upon any of the following factors:

1. The amount of suspended solids in the aerator influent converted to new sludge less the amount lost in the effluent.

2. The degree of auto-oxidation or endogenous respiration.

3. The concentration of suspended solids carried in the aeration tanks.

4. The suspended solids produced by coagulation from dissolved material.

5. The growth rate which is related to the concentration and nutritional value of the waste and the access of the activated sludge organisms to the waste.

W. W. Eckenfelder Jr. and R. F. Weston (Biological Treatment of Sewage and Industrial Wastes - Vol. 1.) gave the general expression:

Biological volatile sludge produced = a (BOD removed) -b (mixed liquor volatile solids) in which "a" represents that fraction of the BOD removed which is synthesized to new biological sludge and "b" represents the mean rate of endogenous respiration expressed as % per unit time.

The fraction of BOD removed which is synthesized to new sludge is not a constant for all sewages. Experiments have shown values of "a" to range from .47 to 1.4 (0.5 average). Also the rate of endogenous respiration is not a constant for all sewages and treatment processes (0.05 to 0.02 or less at lower loadings than 30 pounds per 100 pounds solids).

It is apparent from this expression that the greater the concentration of volatile solids in the mixed liquor the less will be the poundage of volatiles to be wasted, provided the BOD loading remains the same.

It has been observed that soluble BOD is more difficult to process than a mixture including soluble, colloidal and suspended materials having the same oxygen demand. The process of converting dissolved material to suspended solids by biological action is a rather slow one. The amount of new sludge produced in the aeration tanks is expected to be less where the sewage being treated has a high soluble BOD concentration.

Nitrogen is one element required by micro-organisms in the activated sludge process in measurable amounts.

Systems with less than about 8 PPM free ammonia as N will die because more protoplasm is degraded during endogenous respiration than is synthesized during metabolism. In this case a great build-up of polysaccharides, material resistant to biological metabolism, occurs. Because of this increase in polysaccharide material, the level of solids would increase although the system was being destroyed.

In moderate nitrogen systems containing about 8 to 18 PPM free ammonia as N., there is little or no daily increase in true protoplasm and the process maintains itself. Wasting is required however to prevent the build-up of polysaccharide material.

In high nitrogen systems the high demand for synthesis is met by the formation of true protoplasm. Large quantities of protoplasm are formed at the head-end of the aeration tank. Unless sufficient air and detention time are provided to destroy the protoplasm by endogenous respiration a great increase in total sludge mass will occur, necessitating much wasting to prevent bulking and to maintain uniform conditions in the treatment process.

It is felt by some investigators that the sludge accumulation is best expressed in terms of pounds volatile solids per pound BOD applied vs. pounds volatile mixed liquor suspended solids per pound BOD applied.

The Sludge Age has been defined previously as:

$$Sa = \frac{Va \times Ca}{Q \times Cs} \quad \text{where}$$

Sa = Sludge Age in days

Va = Volume of aeration in millions of gallons

Ca = Average concentration of suspended solids in the aerator in PPM.

Q = Rate of sewage flow in millions of gallons

Cs = Sewage or primary effluent suspended solids in PPM

The best sludge age for the conventional activated sludge process on this basis is from 3 to 4 days.

Where the volatile content of the activated sludge is subject to radical changes, it has been proposed to substitute the weight of volatile suspended solids in the aerator rather than the total. Also, where soluble industrial wastes are being treated it has been proposed to determine the Sludge Age based on BOD rather than suspended solids.

Also, if the Sludge Age were expressed in terms of BOD feed and volatile suspended solids content of the aeration tanks, then for a given Sludge Age the sludge accumulation should be the same irrespective of the total mixed liquor suspended solids concentration.

Sludge Wasting

Sludge wasting at some plants is based on an arbitrary percentage of the return sludge flow. This method can be improved by using a sliding percentage scale depending on the concentration of suspended solids in the return sludge.

One plant reported wasting from 3 to 5% by volume of the return sludge when the return sludge solids were in the 7,000 to 10,000 PPM range.

L. S. Kraus (Sewage Works Journal - July, 1959) presented the following formula for sludge wasting:

$$W = \frac{QcI}{10^6} \quad \text{where}$$

W = Waste activated sludge (MGD)

Q = Raw sewage flow (MGD)

c = Suspended solids concentration of activated sludge formed (PPM)

I = Sludge Volume Index

The value of "c" to be used depends on the BOD removal in the secondary treatment units and "a" the fraction of BOD removed which is synthesized to new sludge.

For Example

Sewage Flow	=	1.0 MGD
5-Day BOD in Primary Effluent	=	130 PPM (as before)
Assume 5-Day BOD in Final Effluent	=	15 PPM
5-Day BOD Removed in Aeration Tanks	=	115 PPM
Assume S.V.I.	=	100
Assume "a"	=	0.5
Then c	=	0.5 x 115 = 57.5 PPM

% Waste Sludge of Sewage Flow $= \frac{cI}{10^6} \times 100 = \frac{57.5 \times 100 \times 100}{10^6}$

$= 0.575\%$

In this formula wasting varies directly as the Sludge Volume Index. To maintain proper plant balance secondary sludge wasting to the primary tanks should be increased as the Sludge Volume Index rises.

By establishing specific values for "a" and "b" Heukelekian et al (Sewage and Industrial Wastes Journal - August, 1951) presented the following formula:

$A = 0.5\ B - 0.055\ S$ where

A = LBS. volatile solids accumulated per day

S = LBS. volatile mixed liquor suspended solids

B = LBS. of 5-Day BOD fed per day

For Example

Sewage Flow = 1.0 MGD

Total 5-Day BOD loading to aeration Tanks = 1300 LBS./DAY (as before)

Total Suspended Solids Carried in Aeration Tanks

= 6500 LBS. (as before)

Assume Volatile Suspended Solids = 75%

$$A = 0.5 \times 1300 - 0.055 \times .75 \times 6500$$
$$= 650 - 268$$
$$= 382 \text{ LBS. volatile suspended solids/day}$$

Assume Return Sludge total Suspended Solids = 10,000 PPM
Volatile Solids = 0.75 x 10,000 = 7,500 PPM

7,500 lbs. are contained in 100,000 Gals. of sludge

$$382 \text{ lbs. are contained in } = \frac{100{,}000 \times 382}{7{,}500} = 5020 \text{ GALS.}$$

$$\%\text{ Waste of Raw Sewage Flow} = \frac{5020 \text{ (GALS.)} \times 100}{1{,}000{,}000 \text{ (GALS.)}} = 0.502\%$$

Although this formula is limited in its application since the assumed specific values of "a" and "b" do not apply to all plants, it may be used as a guide to proper secondary sludge wasting.

MIXED SLUDGE QUANTITIES FROM PRIMARY SETTLING TANKS

By neglecting supernatant flow and any gain or loss of solids due to biological activity the theoretical volume of mixed sludge (raw sludge and waste activated sludge) produced in the primary settling tanks (GPD) can be determined from the sewage flow, raw sewage suspended solids concentration, overall suspended solids removal, total solids content of the mixed sludge and the specific gravity of the mixed sludge as follows:

Mixed Sludge (GPD)

$$= \frac{\text{Flow (MGD)} \times \text{Raw Sewage S.S. (PPM)} \times \text{Overall S.S. Removal (\%)}}{\text{Total Solids Mixed Sludge (\%)} \times \text{Specific Gravity Mixed Sludge}}$$

The specific gravity of the mixed sludge (1.03) is only slightly greater than that of water (1.0) and is usually ignored.

For Example

Flow = 1.0 MGD
Suspended Solids in Raw Sewage = 200 PPM
Suspended Solids in Plant Effluent = 15 PPM
Overall Efficiency $= \frac{200 - 15}{200} \times 100 = 92.5\%$
Total solids in mixed sludge = 4%
Specific gravity of mixed sludge = neglect

$$\text{Mixed Sludge (GPD)} = \frac{1.0 \times 200 \times 92.5}{4.0}$$

$$= 4{,}625 \text{ Gallons}$$

The raw sludge volume can be calculated in the same manner by using the suspended solids removal efficiency of the primary settling tanks.

L. S. Kraus (Sewage Works Journal - July 1949) established the following mathematical relationship:

$$P = \frac{\frac{F + B}{8.4}\left(1 - \frac{0.65\, m}{d}\right)}{p - m}$$

where
P = primary settling tank sludge (1000 US gal./hr.)
F = Rate of fresh (raw) solids removed in primary settling tanks (LBS/HR.)
B = Rate of suspended solids formed in activated sludge system (LBS./HR.)
m = Digester liquor S.S. Concentration (%)
d = Waste Digested sludge S.S. Concentration (%)
p = Primary settling tank (mixed) sludge S.S. concentration (%)

This expression not only recognizes that the quantity of mixed sludge to be removed from the primary settling tanks (P) increases with raw sludge removal (F), secondary sludge wasting (B) and decreases with solids concentration (p) but also considers the effect of digester solids concentrations.

The equation derived is in the form of a hyperbola. This means that the rate of mixed sludge flow (P) approaches infinity as the solids concentration of the supernatant (m) approaches the solids concentration of the mixed sludge (p).

It has already been shown that as the Sludge Volume Index increases the volume of waste sludge and therefore the primary sludge increases.

If the system were in balance while pumping 3.5% mixed sludge to the digester and returning 0.5% (5000 PPM) supernatant and a change in SVI reduced the mixed sludge concentration to 2.5% the rate of flow of mixed sludge would increase from 8,000 to 10,000 US GPH. If the supernatant were of poor quality having 2% solids the mixed sludge flow would increase from 14,000 to 30,000 US GPH.

This demonstrates both the adverse effect of returning poor-quality supernatant to the primary settling tanks and sudden changes in mixed sludge solids concentration.

If the flow of mixed sludge to the digester is not increased accordingly by the operator the sludge level in the primary settling tanks will increase and the sludge will ultimately leave the tanks with the primary effluent. This condition will require an increase in the waste activated sludge, which will further add to the load on the primary settling tanks. Thus, there is established a vicious cycle which cannot be remedied without losing activated sludge in the final effluent, wasting activated sludge to a lagoon or tank, and/or discontinuing the return of digester supernatant to the primary settling tanks.

EXHIBIT 1

SUMMARY OF FORMULAE AVAILABLE FOR DETERMINING RETURN SLUDGE QUANTITIES

Formula	Definition of Terms	Assumed Values	% Return Sludge	Tests Required
$\frac{100b}{a + b} = p$	a = sewage flow b = return sludge flow p = % settled volume of mixed liquor	1.0 MGD ? 25%	33%	(a) Two flow measurements (b) One 30-minute settling test Flow and Settled Volume
$(a + b)x = by$	x = mixed liquor suspended solids y = return sludge suspended solids a = sewage flow b = return sludge flow	2,500 PPM 10,000 PPM 1.0 MGD ?	33%	(a) Two flow measurements (b) Two suspended solids tests Flow and Weight of Solids
Nomograph	% settled volume of mixed liquor - mixed liquor suspended solids - Sludge Volume Index -	25% 2,500 PPM 100	33%	(a) One 30-minute settling test (b) One suspended solids test Settled Volume and Weight of Solids
% return sludge $= 100 \frac{(P)}{(P-p)}$	P = % settled volume of return sludge p = % settled volume of mixed liquor	95% 25%	36%	(a) Two 30-minute settling tests Settled Volume
% return sludge $= 100 \frac{(x-Cp)}{y-x}$	x = Mixed liquor suspended solids Cp = primary effluent suspended solids y = return sludge suspended solids	2,500 PPM 80 PPM 10,000 PPM	32%	(a) Three suspended solids tests Weight of Solids

OPERATING DIFFICULTIES

OF ACTIVATED SLUDGE PLANTS

by

C. H. KRETCH

Assistant District Engineer

An Address To
The Ontario Water Resources Commission
Intermediate Sewage Works Operators' Course
Toronto, Ontario
March 6, 1962

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INTRODUCTION

Due to variations in raw sewage characteristics, poor treatment plant design, or faulty operation, several difficulties may develop within the activated sludge process to reduce its purification efficiency or to produce nuisances.

From the operator's standpoint, the plant designer should provide facilities for flexible operation in order that the process may be reasonably controlled under difficult circumstances. Some of the more important features desired are:

1. Duplicate treatment units and equipment;
2. Various unit by-passes, valves and pumping equipment so located that flexible operation is assured;
3. Surplus of air supply and return sludge pumping capacity;
4. Flow measurement devices for raw sewage, return and waste activated sludge, and air volume;
5. An adequately equipped laboratory.

It is the purpose of this paper to discuss the symptoms, causes and correction of some of the more common operating problems associated with the secondary treatment units in the activated sludge process.

SHOCK LOADING

It is generally considered that of the various biological treatment processes now in practice, the activated sludge process is particularly susceptible to "shock loads" and many of the problems encountered when using this process are loosely attributed to shock loadings of one sort or another. In general, any rapidly occurring or immediate change in the chemical or physical environment might be classified as a "shock load" because it may seriously affect the established metabolic pattern of the organisms in the activated sludge.

Hydraulic Peaks

Hydraulic overloading conditions caused by storm water entry into the sewers, intermittent raw sewage pumping schedules or industrial operations will reduce the detention time in the various treatment units and often result in the activated sludge being diluted out of the system.

In municipalities with populations less than 50,000 the fluctuation in flow is usually very marked, with peak rates 3 to 4 times the average rates. This can cause trouble, due to hydraulic and/or organic overloading, especially if the peak rates last over most of the day. The period of low flow may not be long enough to offset the period of high flow, even though the average flow may be lower than the design average. Under these conditions it is imperative that the operator regulate control factors based upon peak rates rather than average rates. Sufficient analytical data should be obtained to determine peak loadings in B.O.D. at times of peak loads in order that a sufficient mixed liquor suspended solids level and air supply volume may be provided.

Organic Overload

Small town industries such as dairies, laundries, canneries and other organic processing industries often discharge slugs of high B.O.D. wastes to the sewer system and due to the increased oxygen requirements, the treatment plant may be upset. Air consumption in the activated sludge process normally 1,000 cu.ft. of free air per lb. of influent 5-day B.O.D. in the case of domestic sewage may be increased to 1,600 cu.ft. per lb. or higher when industrial wastes are present. The usual result of such overloading is insufficient purification of sewage evident as a treatment plant effluent of poor quality. In addition the process may be upset, as illustrated by bulking (poor settling properties).

Return of Digester Supernatant

Digester supernatant disposal in the activated sludge plant is troublesome, especially when the digester is not functioning properly. Supernatant is usually returned to the plant influent where it passes through the entire process. Being well seeded with organisms of anaerobic digestion, it tends to increase the septic action in the settling units. If discharged intermittently during sludge pumping, it throws a heavy load on the secondary process. If the mixed liquor is not in condition to receive this load, the sludge soon becomes grey and septic.

The following points should be kept in mind regarding supernatant handling:

1. Supernatant should be returned as uniformly as possible and selected with lowest possible solids content (0.4 to 0.7%) and the B.O.D. should be less than 500 ppm. The 30-minute settling

test may be used as a rough guide for supernatant quality. The settled volume should be less than 10% and preferably less than 5%.

2. Returning it directly to the aeration section often eliminates difficulties.

3. Returning it to the aeration section during low loading periods is sometimes successful.

4. If the supernatant is returned intermittently, the solids in the mixed liquor must be in condition to receive it (higher in concentration). The D.O. must be carefully watched and increased during the period if necessary.

Toxic Elements

Waters or wastes containing a toxic or poisonous substance disrupt the established physiological condition of the microbial population. Salts of heavy metals such as copper, zinc chromium or nickel and cyanide compounds may upset the process to the extent where a prolonged acclimatization period much in excess of the plant flow-through time is required. A sudden change in the pH can have similar results.

The following table gives suggested maximum safe limits of industrial wastes in composite raw sewage to avoid adverse effects on sludge digestion or secondary treatment

TABLE 1

WASTE	MAXIMUM SAFE LIMIT IN RAW SEWAGE	ILLUSTRATIVE SOURCE
Chromium as Cr (hexavelant)	3 ppm	Tanneries, plating, anadyzing of aluminum.
Copper	1 ppm	Metal plating
Cyanides	2 ppm	Metal plating
Acids and Alkaliis	pH range 6.0-9.5	Chemical manufacturing, or use of chemicals by industries.
Phenol or equivalent	50 ppb	Gas manufacturing.
Formaldehyde	Undesirable in any quantity	Manufacture of antibiotics and plastics.
Oils and Greases	100 ppm	Packing houses, garages wool scouring operations
Zinc, Nickel, Mercury, Lead, Arsenic.	Undesirable; safe limits not definitely established.	Metal and chemical Industries.

Temperature Effects

Temperature has an important effect on the rate of biological activity. The activity of activated sludge organisms increases with rising temperatures up to about 105°F. Temperatures above this will kill the micro-organisms in the activated sludge. Low temperatures such as experienced in extremely cold winter months definitely decrease the rate at which work can be done by the activated sludge. Observations at various plants indicate low temperatures result in a more turbid effluent but less frothing in the aeration tanks than obtained during winter weather. This means that, other things remaining equal, more B.O.D. can be oxidized in the summer months than in the winter months. Higher temperature operation, however, requires closer control primarily as the saturation concentration of dissolved oxygen is less in warm water than cold water. It is noted that the optimum temperature for oxidation of activated sludge is about 82°F.

AERATION TANK FROTHING

The formation of a thick layer of froth over the surface of aeration tanks is becoming a more common and more serious problem for the operators of activated sludge plants. The cause (or causes) are not definitely known, though it is frequently attributed to the increasing use in industry and homes of synthetic detergent compounds.

In the smaller plants, the foam builds up during the night when retention periods are long and agitation is extended. In a moderate wind this blows around the plant grounds. It can be a hazard to the operator as it smears the tank walls and walkways with grease and sludge solids, and certainly it is aesthetically undesirable.

Such accumulations interfere with the proper operation of aeration tanks in two ways. First, the blanket hinders the necessary oxygen absorption from the atmosphere by shielding the liquid surface. In diffused air plants this necessitates an increase in the blower output to supply the oxygen deficiency. In mechanical aerators, shielding of the liquid surface from the major source of oxygen could be disastrous. The second interference factor is that foam covering all or almost all of the surface does not permit an operator to make the necessary visual inspection of the air distribution and quality of the mixed liquor.

The quantity of froth formed has been observed to increase with the following:

1. Decrease in mixed liquor suspended solids.

2. Increase in aeration.

3. Increase in degree of purification of sewage.

4. Increase in atmospheric temperature.

Prevention and Cure

All of the control measures now utilized affect a destruction of the foam, preventing its formation, or both. These include raising the suspended solids level in aeration tanks, using chemicals to act on the foam, and beating down the foam with water sprays.

The practice of raising the suspended solids level in the aeration section has been very effective but means altering the original design considerations. The increased solids level results in an increased air demand and the additional power cost may be more than when other methods are employed. This is apparent because the power requirements for aeration in diffused air plants represent 60 to 70% of the total power demand. Furthermore, the air required to maintain the higher solids level may exceed the plant's blower capacity.

Use of "foam-killing" chemicals in aeration tanks is being practiced to a very limited extent in this province. Defoamants are active materials that remove the froth rapidly when applied in small quantities. They are not effective over long periods of time, and repeated dosing, several times per hour, is necessary.

Water sprays have been used considerably in beating down and controlling foam. In many cases recirculation and spraying of the plant effluent has been employed. Both coarse droplets and adequate water pressure (20 to 40 psi.) at the nozzles are required for a spray system to be effective. This method has been successful but is undesirable from the standpoint of wet or icy walkways due to blowing of the spray droplets, and if plant effluents are used, a health hazard is introduced.

Figure 1 illustrates a froth covered aeration tank.

FINAL SEDIMENTATION TANK PROBLEMS

The importance of final settling tanks to the activated sludge process cannot be stressed enough. These tanks must not only produce an effluent of acceptable quality but must separate and deliver a fresh activated sludge to the return sludge pump continuously. Some of the various problems associated with these units are as follows.

Density Currents and Short Circuiting

Density currents are induced in final settling tanks due to the fact that the specific gravity of the sludge mixture is greater than the clarified water in the tank. The activated sludge mixed liquor on entering a settling tank falls almost

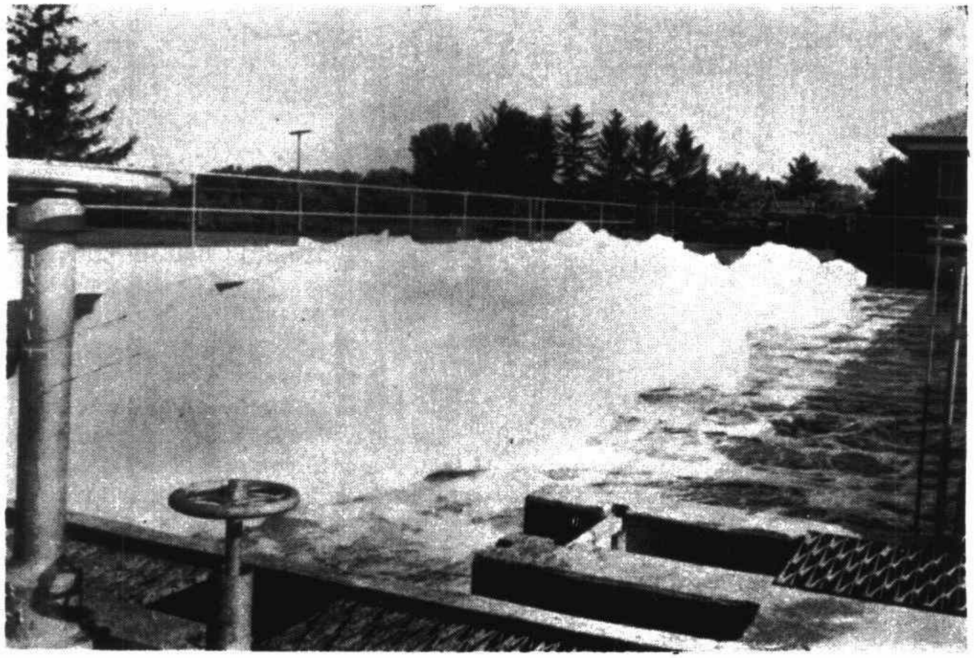


FIGURE I

FROTH-COVERED AERATION TANK

vertically and then flows along the bottom of the tank toward the outlet end or wall. Currents near the bottom of the tank will establish secondary currents in the water layers above them in a reverse direction. This phenomenon leads to a carryover of sludge at the weirs from up-welling currents unless adequate design has been provided.

Diagrams showing the nature of density currents and the effect of turbulence at the base of the tank inlet are shown in Figure 11.

When the velocity of the density current increases until bottom scouring occurs, serious operational problems result such as prolonged detention, deterioration of solids, and carryover of low density particles.

A final settling tank called the "Gould" type has been developed which utilizes the phenomenon of sludge density currents. In these tanks, the mixed liquor is introduced near the tank floor, the sludge collector flights travel in the same direction as the density current, and the effluent weir is located near the inlet end of the tank. This type of unit is illustrated in Figure 111.

Short circuiting through final tanks may be studied by dye tracing and reduced by the judicious use of baffles.

Bulking Sludge

The bulking of sludge is the most baffling problem that confronts the operator of an activated sludge plant. Activated sludge is said to bulk when it settles with great difficulty and its density decreases. Figure 1V shows good and poor settling qualities of activated sludge.

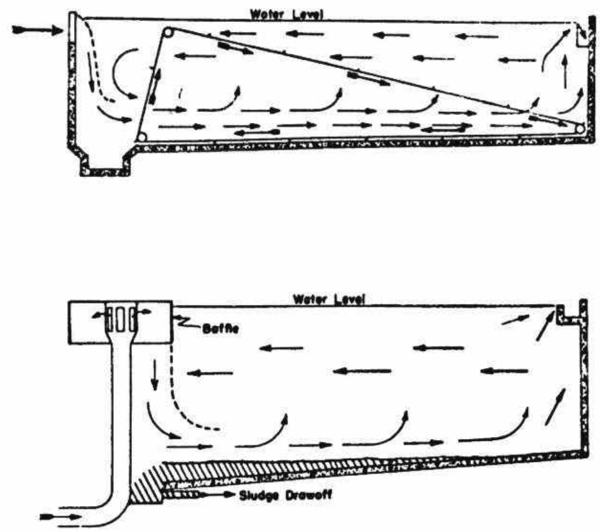
Basically there are two types of bulking. One type is where the sludge is well flocculated but the flocs are extremely large and the sludge volume index is high, usually above 300 for diffused air plants. With this type of bulking, though the floc settles slowly, the supernatant is quite clear.

A second type of bulking is that in which the sludge is slimy, deflocculated, and there seems to be no definite line of separation between the solids and the liquid. Examination of such sludge under the microscope may indicate the presence of the filamentous organism Sphaerotilus. This is illustrated in Figure V which shows photomicrographs of both normal and bulking activated sludge. Masses of these organisms develop into unsightly stringy growths which attach themselves to walls, gates and baffles.

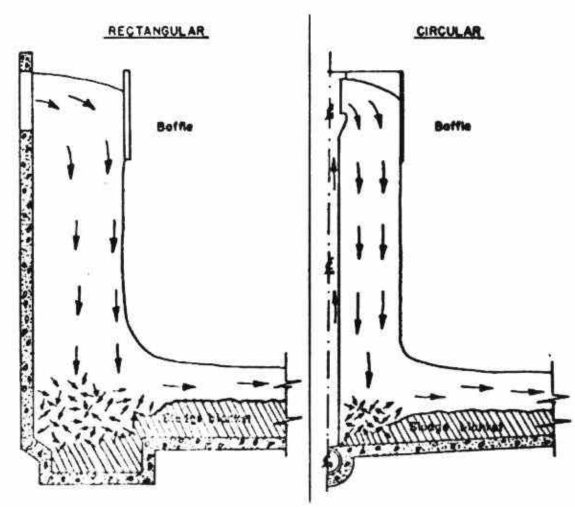
Causes:

Bulking of sludge is attributed by different authorities to a number of causes some of which are listed below:

FIGURE II

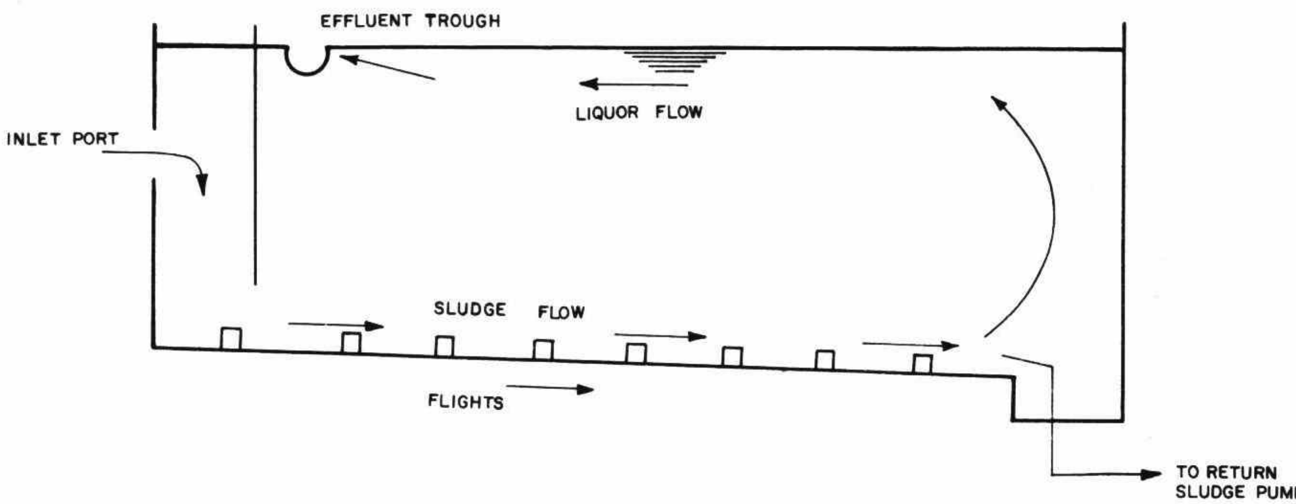


DENSITY CURRENTS AND SECONDARY CURRENTS IN RECTANGULAR AND CIRCULAR CLARIFIERS



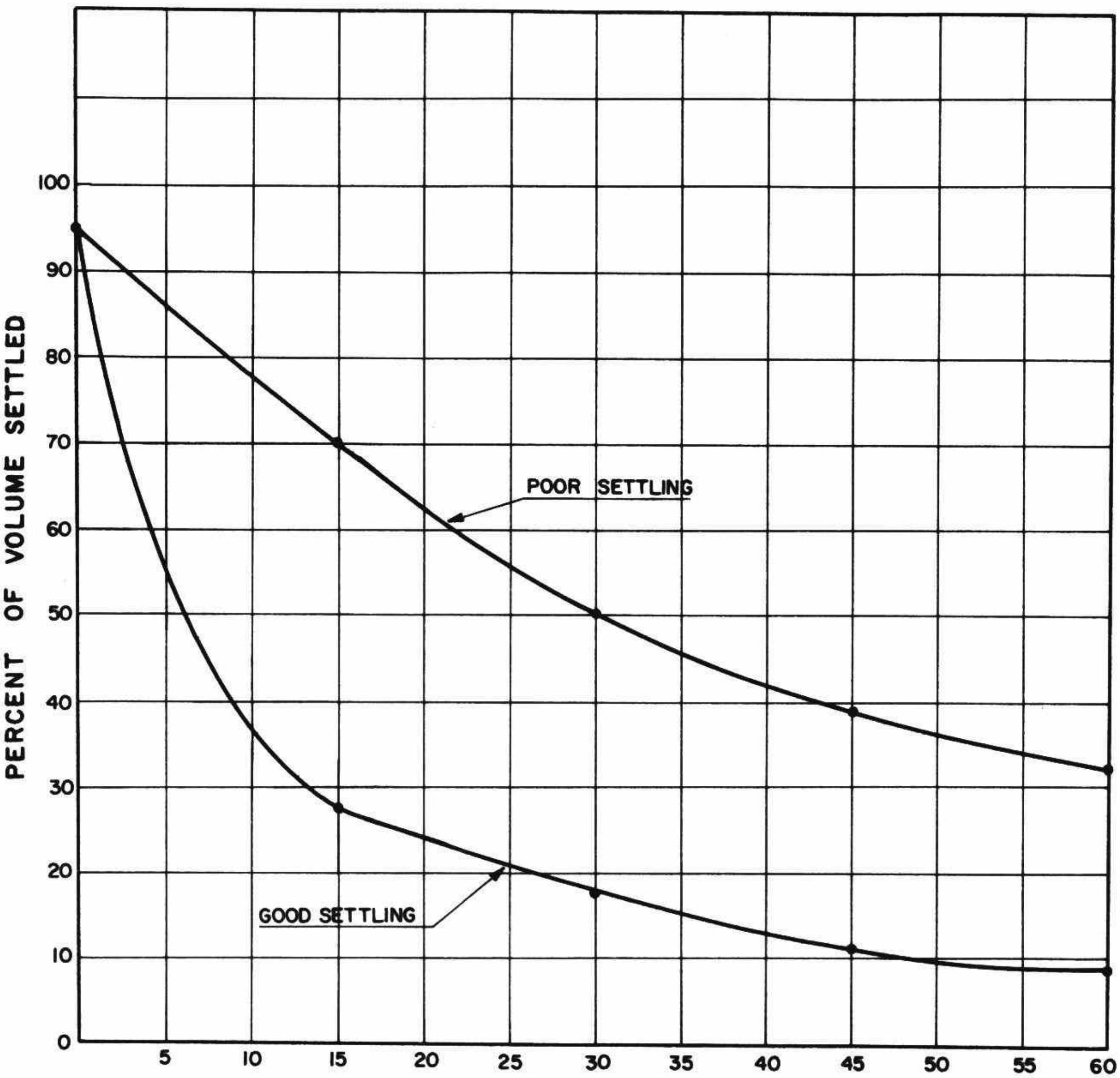
EFFECT OF TURBULENCE AT BASE OF SUBMERGED WATERFALL ON REMOVAL OF THICKENED SLUDGE

FIGURE III



DIAGRAMMATIC REPRESENTATION OF THE "GOULD" TYPE FINAL SETTLING TANK

FIGURE IV



CURVES SHOWING CONDITION OF ACTIVATED SLUDGE

1. Raw Sewage Characteristics:

    (a) Sudden heavy loads on the system such as a heavy dose of strong digester supernatant or an overload of stale or septic sewage solids flushed to the plant by rains after a long dry period.

    (b) Sewage abnormally high in organic solids, especially sugars and starch.

    (c) Harmful industrial wastes present.

    (d) Excessive grease content.

    (e) Low temperature.

    (f) Greatly fluctuating sewage flow.

    (g) Abnormal pH value.

    (h) High iron content.

2. Aeration Tank Operation:

    (a) Inadequate air supply.

    (b) Inadequate aeration period.

    (c) Retarded biological activity.

    (d) Excess suspended solids content in the mixed liquor.

    (e) Poor mixture of activated sludge with the settled sewage.

    (f) Short circuiting through aerators.

3. Final Settling Tank Operation:

    (a) Maintaining excessive depth of sludge.

    (b) Sludge being held too long.

All of these can be summed up by saying that sludge bulking results from overloading or the improper balance between the three variables - incoming B.O.D. load, suspended solids concentration of the mixed liquor, and the amount of air used in aeration.

Correction:

If a case of bulking has been brought on by a heavy load of some organic waste, these considerations must be borne in mind to remedy the situation.

To overcome the overload it is necessary to return more well activated sludge, but as the bulking sludge is neither as well concentrated nor as properly activated as it was before bulking, the volume of return sludge must be greatly increased.

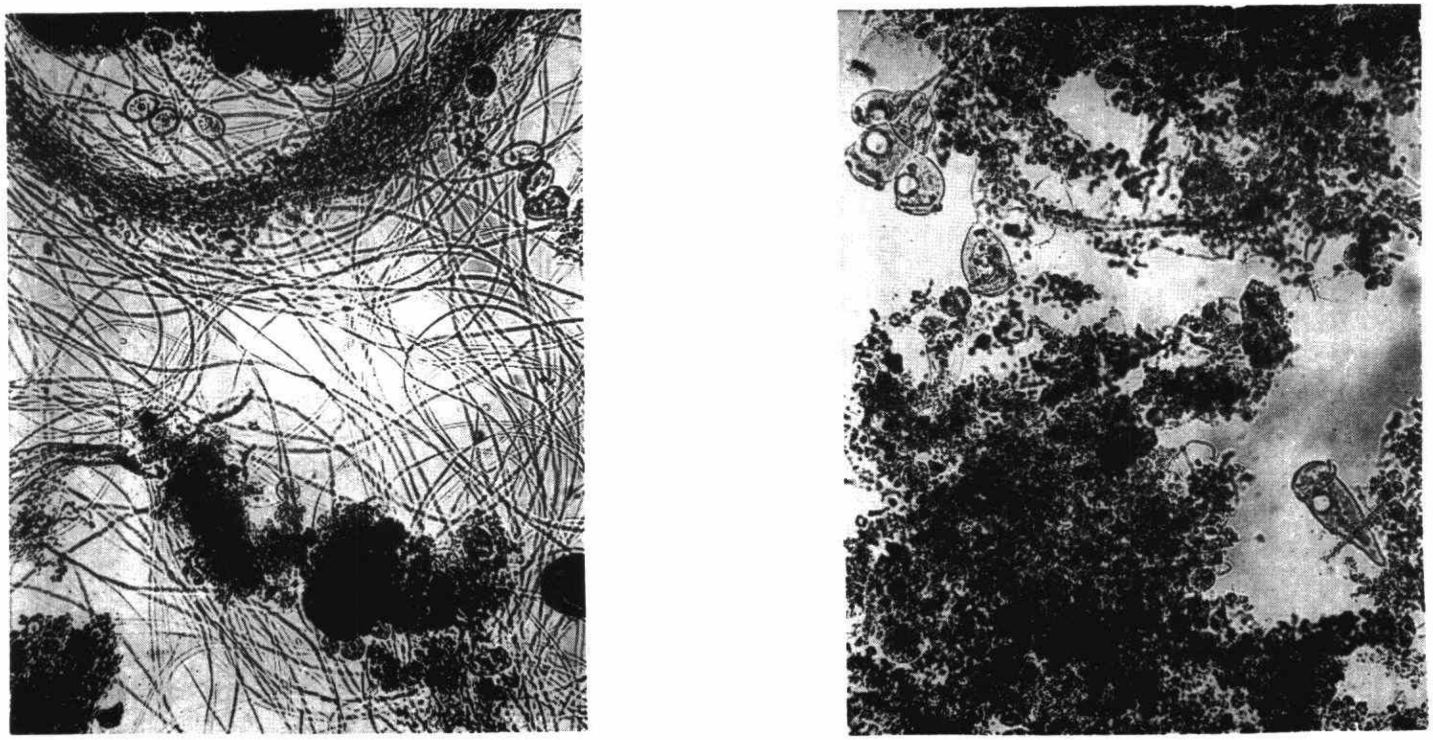


FIGURE V PHOTOMICROGRAPHS OF ACTIVATED SLUDGE
BULKING SLUDGE CONTAINING SPHAEROTILUS ON LEFT, NORMAL SLUDGE ON RIGHT.

There have been many cases when using all the return sludge pumping capacity, only half the required amount could be returned. It is not enough to increase the return sludge and the amount of air in such bad cases of overload and resulting bulking, for the detention time in the aeration tanks is a third factor in the activated sludge treatment process. Sewage of above normal B.O.D. needs increased treatment time in addition to higher sludge and air concentrations. If the air supply cannot be increased, it would be better to reduce the amount of sludge returned, and have a poor effluent, until the load is reduced.

In severe cases of overload and resulting bulking, it will probably be necessary to by-pass at least part of the flow, preferably during the time of heaviest load. If the condition has gone very far, the best remedy may be to remove as much as possible of the bulking sludge from the aeration and final settling tanks and then to develop fresh and properly activated sludge.

Chlorination Treatment

Where bulking has been due to oxygen deficiency in the aeration section, mild chlorine application to the recirculated sludge has been successful in many instances. Chlorination controls the sludge bulking by lowering the sludge volume index to a normal level. The amount of chlorine to be applied to the return sludge line can be calculated from Tapleshay's formula which is:

$$C.D. = S.I. \times F. \times W. \times 0.0000834$$

where

C.D. = Chlorine dosage - pounds per day.
S.I. = Mohlman sludge volume index.
F. = Return sludge rate - M.G.D.
W. = Suspended solids in return sludge - ppm.

Example:

Assume S.I. = 300, F=0.5 and W=4,000
Then Chlorine dose C.D. = 300 x 0.5 x 4,000 x 0.0000834
= 50 pounds per day.

For best results, the rate of returnsludge throughout the clorine dosing period should be kept at a constant percentage of the flow. The chlorine dosage should be introduced to the return sludge at a point which will allow at least two minutes contact period before it mixes with the primary effluent. Where it is not possible to introduce the chlorine into the return sludge line, it can be applied to the sludge hopper of the final tank.

As chlorination is continued the index drops rather rapidly, the solids build up and the turbidity of the effluent decreases. Usually a decided improvement in the index is apparent within 3 to 4 days. This remedy must be used carefully and properly and only when all regular adjustments of the process have failed to bring about an improvement. Overchlorination of the return sludge may kill the organisms in the sludge.

Rising Sludge

Rising sludge is usually correlated with high nitrites and/or nitrates. In the absence of oxygen at the bottom of the final settling tank, denitrification may occur with the release of small bubbles of nitrogen which attach themselves to the floc and cause it to rise. A reduced aeration period and prompt removal of sludge from the final tanks are ways of correcting this condition.

Rising sludge may also be caused by carbon dioxide buoyancy if the sludge has been permitted to become septic. The best remedy is prompt return of sludge to the aeration tank. An increase in the aeration period is required to overcome carbon dioxide buoyancy.

Poor Effluent Clarification

Poor clarification of the effluent is associated with over-oxidation. Much of the sludge is destroyed by the over-oxidation and a "pin-point" floc which is very small and compact develops. Such a sludge does not produce a sparkling clear effluent although nitrification may be high and the sludge has a low index. These particles are very light and do not settle readily. This condition has been observed in plants which are under loaded.

SLUDGE RECIRCULATION AND WASTING

The volume of activated sludge returned from final settling tanks to aeration tanks normally ranges from 20 to 40 percent of the raw sewage flow. A high rate of return reduces aerator detention but keeps sludge fresh and may return needed dissolved oxygen to the aerator inlet. Excessive rates of recirculation tear the floc apart, which decreases its ability to settle. A low rate of return increases aerator detention time and is feasible when the sludge has a low rate of oxygen utilization and does not readily become septic.

Pumping of return sludge may be achieved by constant or variable speed, electrically driven centrifugal pumps or by air lift arrangements. Some difficulties have been reported when adjustments are made to an air lift system. Often when operators adjust the air volume to decrease the return sludge rate, the lines clog. Clogging has been attributed to leaves and other foreign matter which either falls or is blown into the tank. To prevent leaves from entering the tank a covering of chicken wire or similar material is recommended.

The ratio of return sludge to raw sewage flow should be slightly in excess of the volume indicated by the 30-minute settling test conducted on the mixed liquor. Another guide is that the activated sludge solids should be approximately 10 times the weight of the incoming suspended solids from the primary clarifiers.

Example

Assume a flow from the primary clarifiers at the rate of 1.5 M.G.D. having 125 ppm suspended solids.

Pounds of suspended solids to the aeration tanks per day = $1.5 \times 10 \times 125 = 1875$

If the suspended solids in the return sludge are 4,500 ppm, then

$$\frac{1875 \times 10 \times 1,000,000}{4500 \times 10} = 417,000$$

gallons should be returned. This amounts to about 27.8% of the daily flow.

As far as waste sludge disposal is concerned, routine schedules must be developed for wasting in small amounts daily, holding the solids in the aeration tanks nearly constant. Sludge should be wasted slowly and uniformly, generally during periods of low flow.

When attempts to settle the activated sludge in the primary settling tanks fail, a very light primary sludge or a very heavy effluent from the primary tank results. Attempts to thicken it by reducing the pumping rate from the final settling tanks invite septicity of the sludge and bulking from non-active return sludge. It has been found possible to thicken activated sludge in special settling tanks by adding a small amount of chlorine, which prevents septicity, and then drawing off the supernatant liquor to the treatment tanks with the raw sewage and the thickened settled sludge to the digesters.

CONCLUSION

It is of utmost importance that the operator review his procedures periodically to ensure that conditions are being maintained for optimum biological activity in the activated sludge process. Operational procedures should be reviewed to determine the following points.

1. Are samples representative of actual flow conditions? Are they taken at a time of day which will be of most value? Are they composited according to flow?

2. Is sludge withdrawal from the primary tank proper? Is the waste sludge rate controlled adequately?

3. Are aeration tanks operated properly? Is the air supply varied in accordance with the load? Is the mixed liquor concentration proper for the type of plant and load? Is the D.O. level adequate?

4. Are the final tanks operated properly? Is the return sludge rate proper for conditions? Is the D.O. level adequate?

5. Are the digesters operated properly? Is the supernatant returned slowly or all in one slug?

If all of the above points are found to be correct when compared to normal operating figures, no serious problems should be encountered. When, however, one or more vary from the normal, corrective measures must be started, and started immediately.

OPERATION & MAINTENANCE

OF SEWAGE PUMPING STATIONS

by

P. M. HIGGINS

Project Engineer

An Address To
The Ontario Water Resources Commission
Intermediate Sewage Works Operators' Course
Toronto, Ontario
March 6, 1962

OPERATION & MAINTENANCE OF SEWAGE PUMPING STATIONS

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INTRODUCTION

Sewage pumping stations are an integral part of any sewerage system. Only in very rare instances, when all sewage in the collection system flows by gravity, may pumping stations be omitted from the system. It is a fortunate municipality indeed, whose system flows to a central point without the assistance of pumping stations.

I think it would be wise at this time to run through some of the basic design criteria used in planning a pumping station. I'm sure most of you have often said, "why is that station placed there?", or "why is it so deep?", or "why does it pump 400 gallons per minute, when 200 gpm seems adequate?". These are some of the basic questions a consultant will ask himself when laying out a pump station and he will probably answer them thusly:

1. Location: The station is located to take maximum advantage of the natural drainage of an area. However, often a consultant must tie in with an existing system and the station may not on first glance appear to be located in the logical place. A little investigation would probably explain why.

2. Depth: The depth of a station is normally determined by the depth of incoming sewers or possibly by sub-surface soil conditions.

3. Capacity: More often than not stations are designed to meet the requirements of some period in the future, not normally less than 10 years hence. The design capacity is usually based on a population figure, say 100 gallons/capita/day. In a town whose expected population in 10 years time will be 5000 inhabitants (all sewage to be pumped by the station),

the capacity of the station would be $\frac{5000 \times 100}{24 \times 60}$ = 346 gpm. x 150% to 200% of ave. for maximum flow. Perhaps only half of the design flow is being pumped in the early stages of operation. This would explain why the pump was off more than on.

A station must always be adequate to meet the maximum flow rate. If this capacity is not provided, overflows must be included in the design. However, overflows are not always feasible, either from a health point of view or due to the lay of the land.

This brings us to the selection of type of station to be employed. This selection is usually based on the monies available to build such a unit.

When a relatively cheap, small unit is required, often the single wet well is used with one or more submersible pumps. This arrangement is obviously cheapest because only one chamber is required.

Where greater capacity is required and capital cost is not quite so critical, the two cell unit is constructed. This consists of a dry well, housing the prime mover and pumping units, and a wet well.

This may be subcategorized into two:

(i) Units having non-clog centrifugal pumps.

(ii) Units employing pneumatic or air ejectors.

Because house connections are normally 4" Ø the smallest pump size should be 4". Since the velocity should be 2.5 feet per second or more to eliminate settlement of organics, the very minimum capacity should be 100 GPM. Since a 3" diameter solid can traverse the bends and rough joints of a 4" tile pipe line, the sewage pump should be capable of handling at least a 3" solid. Pumps which will pass solids of this size normally operate at a peak efficiency between 400 and 800 gpm. Such pumps are inefficient at low capacities. Ejectors are best for capacities below 100 GPM. For average and large capacities, centrifugal pumps require less space than ejectors of comparable capacity. This explains why a system may have both ejector stations and pump stations. Ejectors are used for capacities below 100 gpm and centrifugal pumps for larger size stations.

At this point, I would like to break a station down into components and briefly discuss the function of each.

1. Pneumatic Ejector Station:

(a) The wet well - The wet well or receiving manhole should be as small as possible to prevent septic action from taking place during very low flow. The sole function of the wet well is to make automatic operation possible with simple controls.

The wet well must be large enough to allow at least 3 to 5 minutes to elapse between successive starts of equipment to prevent over heating of the electric motor driving the compressor. Normally an ejector station will contain an air storage tank to reduce the rapid starts and stops of the equipment.

A basket screen should be incorporated on the influent lines to the wet well to catch any large foreign objects.

Wherever possible, a gravity overflow line from the wet well should be provided to prevent basement flooding in time of equipment failure.

(b) The dry well - The dry well houses all of the mechanical and electrical equipment. The dry well should be large enough that all equipment is easily accessible and it should be completely sealed against leakage from the wetwell. The dry well should however be equipped with a sump, to collect any leakage which does occur.

(c) Equipment - The equipment in an ejector station should include, a discharging or receiver vessel (sometimes this is merely the wet well), a source of air (usually a compressor), a compressor motor, an air storage reservoir, a heater, a dehumidifier, a blower and a sump pump.

Wherever possible the ejectors should be installed in duplicate; each having sufficient capacity to handle the maximum flow. In stations such as this, an alternator is required to equalize the wear on both motors and compressors.

Some sort of level sensing device must be employed in the system, in order that the units may be cut off and on automatically.

There are several commonly used devices which indicate wet well level and in turn energize the motor starters.

(i) A bubbler system - a constant air supply bubbles air through a tube passing from the dry well to the wet well and into the sewage. As the liquid level rises in the wet well a back pressure is exerted on the air column activating a pressure switch, set at a predetermined wet well level, which in turn energizes the magnetic starter.

(ii) The simple float switch - a float running up and down in a guide closes or opens a switch as the liquid level rises or falls. The switch energizes or de-energizes the magnetic starter.

(ii) Electrodes - this consists merely of electrodes placed in a vertical column. The sewage acts as medium of conductivity. Some have afixed to their lower end a plastic bulb. The bulb is filled with an electrolytic solution which is forced up the column when the level in the wet well rises. The hydrostatic pressure exerted by the upward force of the sewage causes the electrolyte to rise making contact with the electrodes. There are normally at least two electrodes in this system, the upper one causing the starter to be energized and the lower one being the cut-off.

Each of these level sensing devices has its advantages and disadvantages, but it is wise to know how all three or modifications of each, operate in case you are confronted by one or all of them.

Most ejectors today have two electrodes inside terminating at the desired high and low level points. However, in small stations often just a timer with a high level electrode is employed to operate the ejector. This means that frequently the vessel is being ejected where it is not full of sewage and sometimes even when it is nearly empty.

The components of a station using centrifugal pumps are fundamentally the same as those for an ejector station, with the non-clog centrifugal pumps and motors replacing the ejector vessel, compressor and motor.

The most popular level sensing device for this type of station is the continual "air purge" or bubbler system. This device requires its own air supply (usually a small rotary compressor and air storage reservoir). It is usually mounted in or under the control panel.

The desirability of above ground pumping stations, which have a greater accessibility to equipment seems to be outweighed by the lower installation cost of the "package" underground stations. However, both type of stations are fundamentally alike, and their operating problems will bear like resemblances.

I would therefore like to press on to the topic of Operation and Maintenance.

OPERATION AND MAINTENANCE

As the sewage pumping station is the key to the collection system and also to the success of the treatment plant (eg. no pumping station, no flow, no treatment), it is imperative that these stations be given just as much attention as the plant itself.

Mr. C. Perry, in his lecture on Maintenance Schedules in Sewage Treatment Plants, will discuss in detail "preventative maintenance". This is planned maintenance, with the objective

of removing sources of trouble before they arise. I would like to stress the aspect of "troubleshooting", or the ability of an operator to realize that something is wrong and be able to find the source of trouble without too much delay.

This is obviously going to involve at least daily inspections of the pump station. I have prepared a check list which might be used as a guide in doing a daily inspection. This may be found at the end of the lecture notes.

There are two main areas in a pumping station where trouble can occur. Firstly in the drive unit and pump itself and secondly in the control mechanism.

Pneumatic Ejector Maintenance

Let us first discuss the maintenance of pneumatic ejector stations. This is a fairly simple procedure. As mentioned in the check list, they should be inspected on a daily basis to see whether or not the motor will run, and whether or not it will cut on and off automatically.

If the motor does not start and stop properly, the first thing to do is to check the control system. The first thing to check would be the overload relay heaters; push the reset button. If the motor still fails to start, the fuses should be checked and replaced when necessary. It is quite possible that the electrodes have become fouled with grease and foreign matter, therefore they should be completely cleaned. If the unit is float switch controlled, possibly the float is greased up and will not ride up and down the float guide, this should likewise be cleaned.

If the automatic controls still do not work properly, the relay and switches should be checked to determine whether or not the contacts are functioning properly. It is important that contacts be maintained in first rate operating condition.

If the motor hums but does not turn over, here is a possibility of single phasing or bearing trouble.

Single Phasing

Perhaps one fuse has blown. If the motor were already running, it would continue to do so, with a good possibility of burning itself out in a matter of minutes. If the motor is stopped, it will not start but will hum quite noticeably. Check the voltage between the lines. If there is no reading check the fuses and replace where necessary. Single phasing can occur in the incoming power lines as well as in the internal wiring. Often it can be detected by quite a noticeable odour in the station.

Bearing Trouble

The switch is turned off, and the shaft turned by hand. If the shaft cannot be turned by hand, the coupling between the motor and compressor is disconnected, and then the shaft of the motor turned. If the shaft turns freely, the trouble will be in the compressor.

If both the motor and compressor work properly and the station does not eject sewage, the pressure gauge should be checked in order to determine whether or not the proper pressure is being built up. If it is not, then the trouble will be a leak in the air line. All connections should be checked to see that they are tight and then leaks should be looked for in the line. This can be done with a simple soap and water solution.

If the motor and compressor are both functioning properly pressure is being maintained in the air lines and the receiver is still not ejecting sewage the trouble is likely to be in the check valve in the influent line. It is more likely than not that this flapper valve is not being allowed to close due to an obstruction by some foreign object. The only solution is to dismantle the check valve and clean the valve seat.

Recently an ejector station in a sewerage system was found to be malfunctioning. It was thought the trouble was due to a faulty timer. The motor and compressor were operating satisfactorily but would cut off immediately after starting. It was found that the trouble was a piece of 2 x 4 lodged in the forcemain. This caused a greater dynamic head than that developed by the ejector. Unfortunately this was not discovered by the operator but by the factory representative of the company who manufactured the ejector, who had travelled over five hundred miles to replace the timer.

It may be appreciated, therefore how much can be saved through a little educated troubleshooting.

Centrifugal Pump Maintenance

The following is a list of some of the major troubles which occur in the operation of a centrifugal pumping station.

1. No Liquid Delivered.

   This may be caused by:

   (a) Lack of prime: Fill the pump and its suction pipe completely with the liquid being handled. To rid the casing and piping of air, open the ventcock on the highest point of the casing until liquid flows freely out.

   (b) Discharge head too high: Check all valves in the discharge line to see that they are fully open.

Check all flapper valves to see that they are not being held shut by some obstruction. If the discharge head is still too high check the discharge line for scale or obstruction.

(c) Suction lift too high: Check the pump inlet for clogging by mud or some other foreign obstruction.

(d) Impeller plugged: Solids may have accumulated on the impeller, preventing its rotation.

2. Pump Vibrates Excessively.

(a) Rough operation: Check to see that the impeller and bowl passages are free of wood, rags, sand and other materials which might throw the pump out of balance. Check the prime mover (or motor) by disconnecting it and operating it alone.

(b) Bearing troubles: Check alignment of the pump with its prime mover.

3. Automatic Controls not functioning properly.

This was discussed in conjunction with the pneumatic ejectors.

Included at the end of the lecture notes are two charts listing some of the more common troubles occuring with centrifugal pumps. These charts may prove useful for rapid reference.

Finally remember the old adage "the chain is only as strong as its weakest link". This is applicable here. Your sewage treatment plant can only operate effectively if the collection system is functional. If a key pump station breaks down, the treatment plant suffers accordingly.

If some minor component of the lift station fails, it is quite possible that this will effect the operation of the whole station. Therefore it is essential that every part of a lift station serves the purpose for which it was designed. It is up to the operator to see that this is so. The ability to spot trouble when it exists and before it becomes serious is very important.

Finally remember that cleanliness of a station is important. "A clean station is quite often a trouble-free station".

TYPICAL DAILY CHECK LIST

Pneumatic Ejector Station

1. Start compressor motor manually to see that it is functioning properly.

2. Alternate compressor units.

Centrifugal Pump Station

1. Start pumps manually and alternate.

2. Check seal assembly - if mechanical seal.

    (a) no leakage should occur through the seal.

    (b) seal water should be clean - open vent cock on seal water line to see that seal is actually getting sealing fluid.

    If conventional packing.

    (a) check for excessive leakage - if this occurs replace packing or tighten packing gland. Tighten gradually and evenly. Packing gland should be just finger tight.

General

1. Lights and blower should operate. In the case of underground lift stations the lights and blowers should operate when entrance hatch is open.

    Make sure manual switch is off when leaving station as excessive ventilation will cause condensation.

2. Check level sensing device to see that units cut on and off at the predetermined levels.

3. Check sump pump operation by lifting float.

4. Check dehumidifier for icing. If the coil is frosted up, turn off dehumidifier until it is defrosted.

5. Check the thermostat on the heaters and maintain the temperature at around 45° F.

6. Check circuit breakers and overload re-sets, to make sure they are on.

7. Where a standby unit is provided start it daily to see that it functions properly and keep gas tank filled, and battery topped up.

8. Finally, touch up rust spots and keep pumping station <u>clean</u>.

    A <u>clean</u> station is normally a trouble-free station.

NO WATER DELIVERED	Section IV
CAUSES	CURES Page 20
1. Lack of prime	Fill pump and suction pipe completely with water.
2. Speed too low	Check power unit.
3. Discharge head too high	Check pipe friction losses. Larger piping may correct condition. Are valves wide open?
4. Suction lift too high	If no obstruction at inlet, check for pipe friction losses. However, static lift may be too great. Measure with mercury column or gauge while pump operates. If static lift is too high, water to be pumped must be raised or pump lowered
5. Impeller completely plugged	Remove top of pump casing and clean impeller.
6. Wrong direction of rotation	This sometimes occurs ! Compare turning of motor with directional arrow on pump casing.
7. Broken shaft or coupling	Replace

NOT ENOUGH WATER DELIVERED

8. Air locks in suction piping	If liquid pumped is water or other non-explosive- and explosive gas or dust are not present- you can test flanges for leakage with flame or match. For such liquids as gasoline, suction line can be tested by shutting off or plugging inlet, putting line under pressure. Seal leaks.
9. Air leaks in stuffing box	Check to see if thin stream of water flows from stuffing box while pump operates, if not- and adjusting gland to reasonable extent does not produce flow- new packing probably is needed Or- water seal piping may be plugged and need cleaning out; lantern ring may be displaced and need centering at water seal piping.
10. Speed too low	See item 2, above.
11. Discharge head too high	See item 3, above.
12. Suction lift too high	See item 4, above.
13. Impeller partially plugged	See item 5, above.
14. Worn wear rings	Inspect. Replace if worn excessively.
15. Worn impeller	Inspect. Replace if damaged or blades eroded.
16. Worn packing	Replace packing and sleeves, if badly worn.
17. Foot valve too small or partially obstructed	Inspect. Area through parts of valve should be at least as large as areas of suction pipe -preferably $1\frac{1}{2}$ times. If strainer is used, net clear area should be 3 to 4 times area of suction pipe.

NOT ENOUGH WATER DELIVERD (CON'T)

CAUSES	CURES
Wrong Direction of rotation	Symptoms are an overloaded drive and about 1/3 rated capcity from pump. Compare turning of motor with directional arrow on pump casing.

NOT ENOUGH PRESSURE

19.	Speed too low	See item 2, above.
20.	Air or gases in liquid- e.g. marsh gas in swamp water (Test in laboratory, reducing pressure on liquids to pressure in suction line. Watch for bubble formation)	May be possible to overrate pump to point where it will provide adequate pressure despite condition. Better to provide gas separation chamber on suction line near pump, and periodically exhaust accumulated gas.
21.	Mechanical defects	See items 14,15,16 above.
22.	Too small impeller diamter (probable cause, if none of above)	Check with your pump manufacturer to see if larger propeller can be used. Otherwise cut pipe losses, or increased speed- or both, as needed. But be careful that you don't seriously overload your drive.
23.	Incomplete priming	Free pump, piping and valves of all air. If high points in suction line prevent this, they need correcting.
24.	Suction lift too high	See item 4, above.
25.	Air leaks in suction piping	See item 8, above.
26.	Air leaks in stuffing box	See item 9, above.
27.	Air or gases in liquid	See item 20, above.

PUMPS TAKES TOO MUCH POWER

28.	Head lower than rating, pumps too much water	Turn down impeller's outside diameter in amount advised by your pump manufacturer.
29.	Liquid heavier (in either viscosity or specific gravity) than allowed for	Use bigger motor.
30.	Wrong direction of rotation	See item 18, above.
31.	Stuffing boxes too tight	Release gland pressure. Tighten reasonably. If sealing water does not flow while pump operates, replace packing. If packing is wearing too quickly, replace second shaft sleeves and keep water seeping for lubrication.
32.	Casing distorted by excessive strains from suction or discharge piping	Check alignment. Examine pump for friction between impeller and casing, worn wearing rings. Replace damaged parts.
33.	Misalignment	Realign pump and motor.

GAS COLLECTION AND UTILIZATION

by

A. C. BEATTIE

Project Engineer

An Address To
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Fundamentals of Gas Production

Raw sludge contains volatile (organic) and non-volatile (inorganic) material. When it is confined and maintained in an airless environment at a proper and reasonably uniform temperature, the volatile material will be acted upon by microorganisms normally present. As a result of this action, or digestion, sludge gas will be generated.

Carbohydrates such as cellulose, starch, and other polysaccharides, proteins and amino acids, fats, soaps, fatty acids, and alcohols have been found to be good sources of methane, but these findings do not necessarily prove these compounds to be the primary source of methane. Studies with cultures of methane organisms have shown that they do not utilize such substances as cellulose, glucose, proteins, amino acids and fats, but that they do utilize a restricted group of simple compounds consisting of lower fatty acids (formic, acetic) and alcohols (methyl, ethyl). The transformation of complex organic materials is thus apparently brought about in two stages by two different groups of bacteria. The complex organic materials are converted by a variety of common bacteria to volatile acids and alcohols without the production of methane; these products are then converted by a restricted and specialized group of bacteria.

CONSTITUENTS OF GAS

Sludge gas usually contains 60 to 75 per cent of methane, 15 to 30 per cent of carbon dioxide, 1 to 10 per cent of nitrogen, small quantities of hydrogen sulphide and sometimes small amounts of hydrogen and oxygen.

The heat value of the gas varies with operating procedures, but most often falls in the range of 600 to 700 BTU per cu. ft. under standard conditions. This compares to values of approximately 1000 BTU per cu. ft. for natural gas, 500 to 600 BTU per cu. ft. for manufactured gas, and 140,000 BTU per gallon for No. 2 fuel oil.

No. 2 fuel oil can be purchased in large quantities for 14¢ per gallon, therefore, the value of sludge gas is $\frac{140,000}{14 \times 600} = 16.6$ (17) cu. ft. per 1¢.

During digester start-up or following periods of overloading, addition of toxic materials, or substantial temperature changes, the methane content will be reduced and the carbon dioxide will increase correspondingly and the hydrogen sulphide content may become a serious problem.

RATE OF GAS PRODUCTION

The daily rate of gas production can be expressed in the following ways:

Primary sludge: 0.5 to 1.0 cu. ft. per capita per day
or 6 to 10 cu. ft. per pound of volatile matter in the raw sludge per day
or 15 cu. ft. per pound of volatile matter destroyed per day

Example: 10,000 persons
S.S. in sewage 200 ppm
Flow = 100 gallons per capita per day = 1 MGD

Primary Plant.

50% revoval of solids

∴ Pounds of solids removed = 10 x 100 = 1000 lbs.

70% volatile matter in sludge

∴ Pounds of volatile matter added = 700 lbs.

Volatile matter is 50% destroyed during digestion

∴ 350 lbs. are destroyed

Gas Production Per Day

∴ Cu. ft. per capita = 0.5 to 1.0 x 10,000 cu. ft.
= 5,000 to 10,000 cu.ft.

Cu. ft. per lb. volatile matter in raw sludge
= 6 to 10 x 700 = 4200 to 7000 cu. ft.

Cu. ft. per lb. volatile matter destroyed
= 15 x 350 = 5250 cu. ft.

Minimum and maximum hourly rates of gas production may be from 45 to more than 200%, respectively, of the annual rate of production. Continuous or intermittent mixing of digesters often aids in providing uniform gas production.

GAS COLLECTION AND STORAGE

There are two common types of digestion tank covers which are used to store gas,

1. A fixed concrete or metal roof which usually has a projecting metal box for collecting the gas.

2. A floating metal roof which is dished to direct the gas to a central collecting dome or cylinder and which has sides extending downward to form a skirt which contains the gas. This cylinder encloses the gas draw-off pipe and allows the cover to rise and fall with the liquid level in the tank without interference with the piping. The travel of the cover is usually restricted to the upper third of the tank depth by means of brackets attached to the tank walls.

Normal operation of a floating cover

As the liquid level rises in the tank, the gas holder rises with it until the liquid overflows the overflow regulating pipe. For proper operating control the elevation of the top of the regulating pipe must be above the maximum high water level to be maintained in the tank by a number of inches corresponding to the gas pressure for which the digester is designed. For example, if the digester is designed for 6" of gas pressure, the regulating pipe must be adjusted so that the overflow is 6" higher than the maximum liquid level.

When the gas holder reaches its maximum gas capacity and correspondingly its maximum lift, it comes against stops on the inside tank wall which prohibit further rise of the holder. Additional gas formation in excess to that being used will then cause the pressure to increase. When the pressure rises about 2" above the natural pressure (that at which the holder floats freely) the waste burner valve (if a waste gas burner is used) should be set to open and thus burn the excess gas. As soon as the pressure is relieved, the waste burner valve automatically closes again, In case the waste burner valve is defective, the gas line becomes plugged, or the pressure rises excessively through some other cause, the pressure vacuum relief device should be set to open at approximately 4" above the natural pressure and vent the excess gas to the atmosphere. It is important that the valves under the pressure vacuum relief devices are kept open during all normal operation.

When withdrawing sludge from the tank a safe rule to follow is to stop as soon as the gas holder comes to rest on the corbel supports. If more sludge is withdrawn after the holder is resting on the corbels, a partial vacuum may be formed in the holder causing the relief device to open and allow air to enter the holder.

Separate Storage Tanks

Gas holders with or without gas compressors are used when sludge gas is utilized for gas engines or incinerating purposes. Holders adjust for the difference in rate of gas production and gas consumption, maintain uniform pressure, and are desirable at many plants to store peak rates of gas production for use in gas boilers and other gas utilization equipment at times of lower gas production. If gas storage is practiced, it is desirable to provide a storage capacity of at least 25 to 50% of the average daily gas production.

High pressure gas storage tanks are usually in the shape of a sphere in diameters ranging from 20 to 70 ft. The gas is compressed and stored from 25 to 50 psi. High pressure holders permit two or three volumes of gas to be stored per unit of tank volume. Precautions against moisture accumulation and its effect on the operation and maintenance of the tank must be observed.

GAS PIPING

The main gas collector from the digestion tank is usually at least 2 1/2 inches in diameter with the gas intake being well above the digester scum level. Because of the high peak rates of gas production often encountered, the present tendency is to use larger minimum sizes than 2 1/2 inches diameter, even to sizing the piping for three to four times the average gas production rate. Sludge gas collection systems for large cities may require gas pipes eight inches or more in diameter. The maximum velocity in sludge gas piping is usually limited to not more than 11 to 12 fps. This velocity will prevent the carry-over of condensate from the condensate traps. Freezing of condensate in the gas may be avoided by running gas pipes from inside the digesters directly to operating galleries, or underground below the frost line. Gas pipes exposed on digester covers are sloped for drainage and may require insulation.

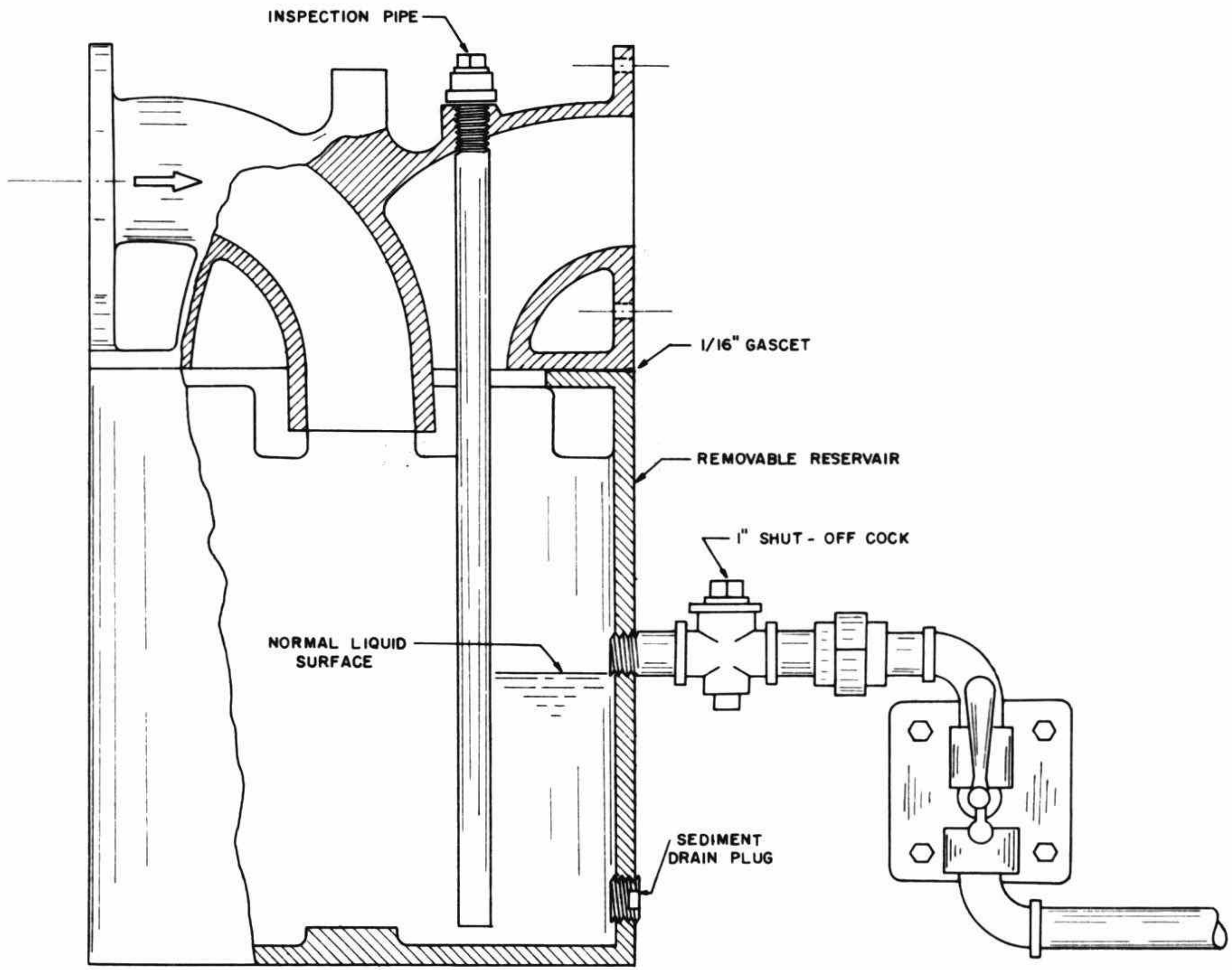
Gas pipe slopes of 1/2 inch per foot are desirable with the minimum slope of 1/8 inch per foot for drainage. Adequate pipe support is essential to prevent breaks. Special care is required where gas pipes are located underground as sags or breaks can cause water pockets and stoppages. A combination of flexible pipe joints and adequate pipe supports prevents breaks. A valved pipe by-pass is considered desirable for gas meters but by-passes are not provided for pressure relief devices or flame traps.

GAS PIPING APPURTENANCES

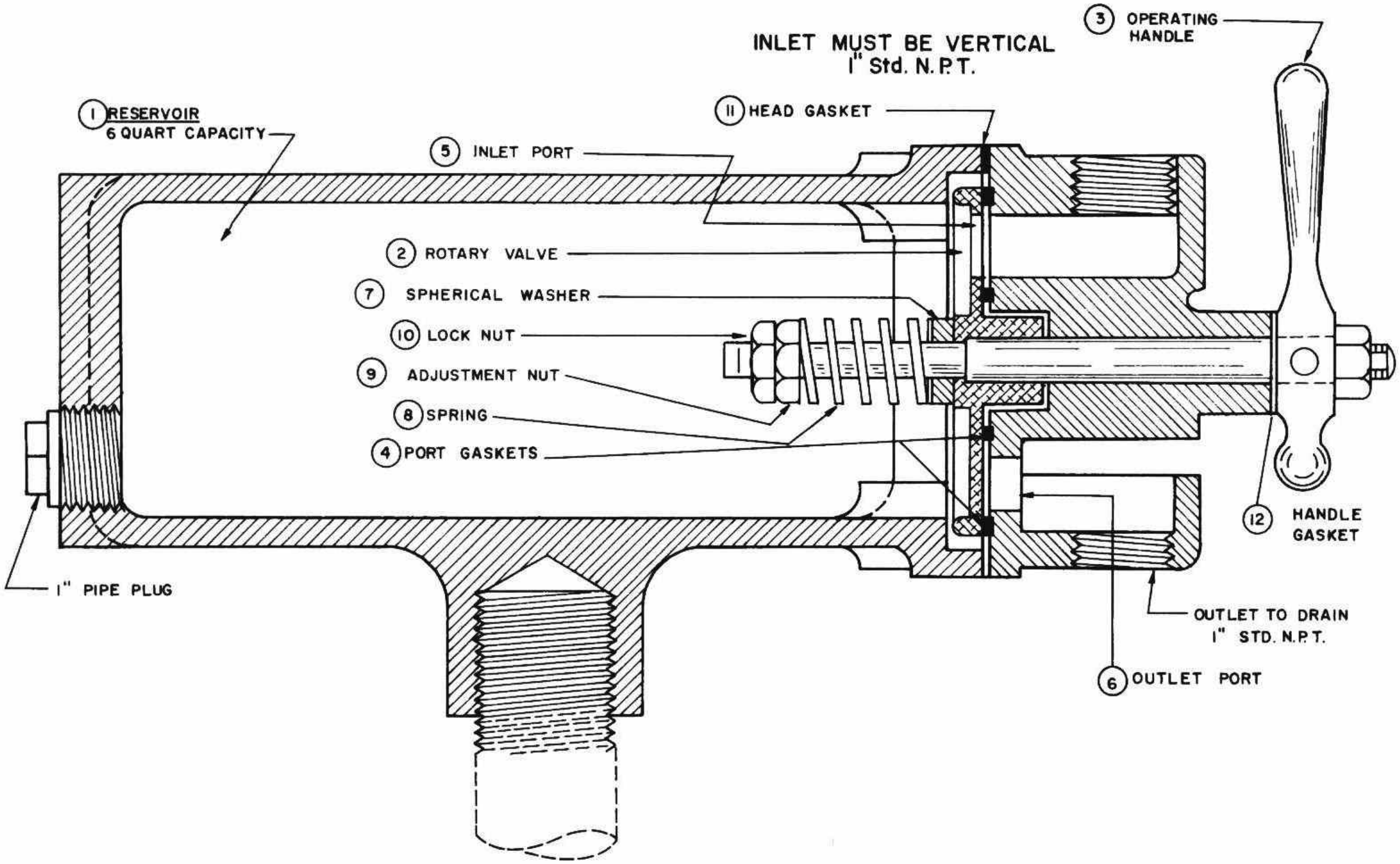
Sediment, Condensate and Drip Traps

The main gas take-off from the digesters is usually pitched to a sediment trap installed at the low point in the system. Sediment traps remove scale and other impurities in the gas together with varying amounts of sediment.

CONDENSATE AND SEDIMENT ACCUMULATOR



DRIP TRAP


INLET MUST BE VERTICAL
1" Std. N.P.T.
3 OPERATING HANDLE
1 RESERVOIR
6 QUART CAPACITY
11 HEAD GASKET
5 INLET PORT
2 ROTARY VALVE
7 SPHERICAL WASHER
10 LOCK NUT
9 ADJUSTMENT NUT
8 SPRING
4 PORT GASKETS
12 HANDLE GASKET
1" PIPE PLUG
OUTLET TO DRAIN
1" STD. N.P.T.
6 OUTLET PORT

Sludge gas, as produced, is wet and dirty. Some bacterial slime may be carried along with the gas, and in addition, the hydrogen sulfide content may be great enough to react with metals in piping and equipment. Consequently, periodic flushing of the gas piping may be required to remove corrosion products and slime growths. As the gas passes into cooler areas, part of its water content is removed by condensation, and this water collects in the piping. It is essential that safe, convenient condensate traps be provided to remove this water.

Drip traps should be installed at all low points in the gas piping system. Manually operated drip traps are preferable. Float controlled drip traps may be used, but, as floats sometime become stuck open, the drains are vented with tight piping connections to the outside of any structure. Generally, float controlled drip traps are only used outside.

GAS METERS

Separate meters to measure gas production from each digester should be used along with a waste gas meter. One meter for total gas production is usually installed for digesters in series. Metering the gas produced offers an immediate indication of some upset or change in the normal progress of digestion. Provisions for lubrication and removal of condensate from the meters are common requirements. Gas meters are particularly vulnerable to the damaging effects of wet, dirty sludge gas. They must be cleaned and drained regularly.

MANOMETERS

Manometers, indicating the gas pressure in inches of water, are usually used at the following locations to permit intelligent operation of the gas collection system:

1. On the main gas line from the digesters.

2. After gas meters, flame traps, and pressure regulators to indicate pressure drops,

3. Just ahead of gas utilization devices.

4. On the open feed line to waste burners to indicate stoppages. The use of any open end U tubes as manometers in enclosed structures is a hazard of the first magnitude.

PRESSURE RELIEF VALVES

To guard against excessive pressures in digestion tanks "weight" or liquid level" types of pressure relief valves are installed. These devices are usually set to hold the gas pressure between 8 and 16 inches of water column depending on the design pressures of the gas system and digester roofs.

This safeguard is incorporated in tanks to prevent rupture of the top or loss of the floating cover. These valves should be checked once or twice yearly in accordance with the manufacturer's recommendations.

Pressure reliefs should be installed in a horizontal section of the gas piping with the diaphragm (15) above and in a horizontal plane. The pressure relief valve controlled by upstream pressure serves to allow excess gas beyond use requirements to pass to a waste gas burner for disposal.

The vent from the topside of the diaphragm must be open to permit diaphragm movement. It should be connected to a vent leading outside the building to prevent escape of gas into confined areas in the event of a rupture or deterioration of the diaphragm.

The pressure relief is adjustable by adding or removing weights above the diaphragm. Access is attained by removing the diaphragm cover (13). The weight holder (16) and the valve parts 18 and 19) are equivalent to an initial value of 1" W. C. weights and are normally supplied to permit adjustment of 1/4" increments to a maximun of 1" W.C.. below the estimated maximun pressure that will be developed at the gas dome of the floating cover.

For gas holder installations, the pressure relief is generally adjusted to 1/4" W.C. above the normal pressure developed by the gas holder. For special applications, diaphragm castings and weights may be supplied to permit a maximum relief pressure of 15" W.C.

The sheepskin diaphragm should be inspected occasionally to make sure that it is soft and pliable and should be treated with Neatsfoot oil if it has become dry or stiff. The diaphragm must be gas-tight and should be replaced when there is evidence of deterioration. The normal life for a diaphragm will be 3 to 5 years, but this may be considerably less during initial stages of digester operation because of the poor quality of the gas.

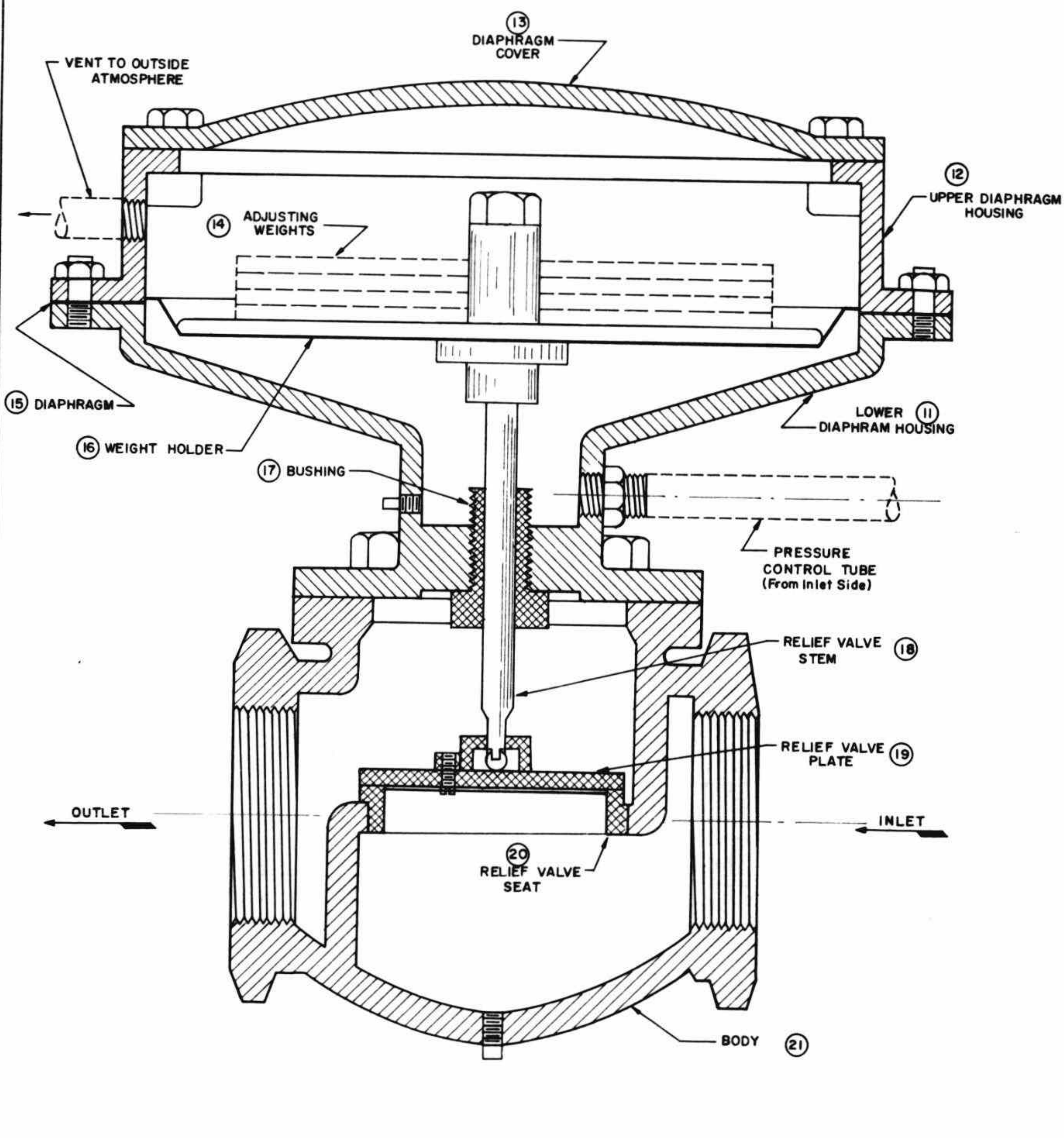
The movement of the stainless iron valve stem (18) should be free through the aluminum bushings (17) and if there is evidence of binding that might be occasioned by an accumulation of foreign matter the valve should be dismantled and cleaned. Before separating the lower diaphragm housing (11) from the body (21), the gas line to the inlet should be closed, as should also the pressure control tube.

The 1/4" pipe plugs at the bottom of the body and at the bottom of the lower diaphragm housing are intended for temporary pressure gauge connections and may be used occasionally to remove small amounts of condensate that are not otherwise removable.

VACUUM RELIEF VALVES

Vacuum relief valves must be installed to prevent undue exterior loadings on the tank cover caused by an uncompensated sludge or gas withdrawal.

PRESSURE RELIEF VALVE



Without vacuum relief under a floating cover, the whole cover might collapse when it sinks to the lower stops and withdrawal of liquid continues. Under these conditions, atmospheric pressure on the top would cause a tremendous pressure on the cover. Similar disastrous results might occur if partially digested sludge should become tightly wedged between the floating cover and side walls so as to make the tank practically gas tight.

The operator should check the vacuum relief assembly once or twice yearly in accordance with the manufacturer's recommendation. It must be remembered that when the vacuum relief valve opens, air is admitted to the digestion tank and an explosive mixture may be created. Therefore, every precaustion must be taken to guard against sparking, open flames, and other sources of ignition.

FLAME TRAPS

Flame traps should always be provided in gas lines directly ahead of gas consuming devices as there is a constant danger of backflashing of fire and possible explosion. The traps usually are provided with bypasses so that they may be removed, inspected, and cleaned of corrosion, dirt accumulation, etc. It is wise to install a flame trap on the bypass lines to provide protection while maintenance work is being performed.

Flame traps should be installed in horizontal sections of gas supply lines to all gas burning appliances. The flame traps afford protection to the gas distrubution and collection system by preventing passage of flame from the locations of gas use.

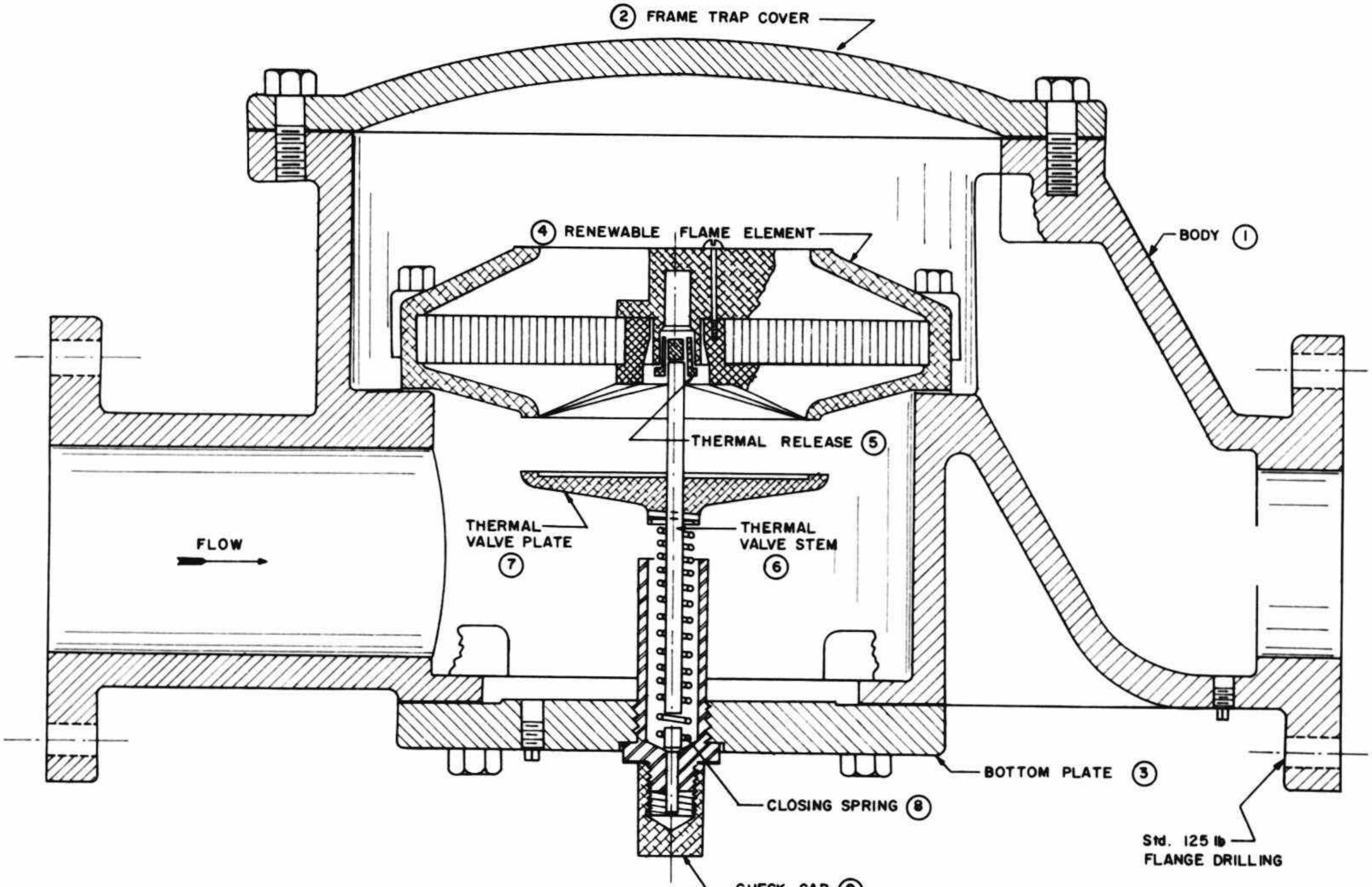
The flame trap consists of a renewable flame element (4) of alternately corrugated and smooth aluminum strips with gas passages sufficiently small and of such length as will prevent passage of flame. The vertical direction of the passages is intended to insure drainage and minimize clogging.

The thermal valve (7) is held open by a thermal release (5) and is closed by a spring (8) working in the direction to the gas flow following continued presence of flame on the topside of the flame element. Continued flame at this location will melt the spider holding a central plug in an annular ring which will permit separation.

When there is reason to suspect that the valve may be closed as evidenced by a cessation of gas flow, the position of the thermal valve may be determined by using the feeler plug after removing the check cap (9). There is 1/8" normal clearance but if the plug may be pushed up until it is flush with the hole from which it protrudes, indication is that the thermal release has separated.

The inlet line should be closed before removing the flame trap cover (2) or bottom plate (3).

The flame element should be inspected occasionally to make sure that the passages are open to permit the free flow of gas.


② FRAME TRAP COVER
④ RENEWABLE FLAME ELEMENT
BODY ①
THERMAL RELEASE ⑤
FLOW
THERMAL VALVE PLATE ⑦
THERMAL VALVE STEM ⑥
BOTTOM PLATE ③
CLOSING SPRING ⑧
CHECK CAP ⑨
Std. 125 lb FLANGE DRILLING

FLAME TRAP

WASTE GAS BURNER

Most systems are equipped with a waste gas burner. Its purpose is to prevent the escape of unburned gas into the atmosphere, thus preventing odours and avoiding the hazards which might result from the escaping gas. The relief valve to the gas burner should be set at the proper pressure so that excess gas goes to the burner instead of blowing off through the pressure relief located on the digester. Care should be taken to keep the pilot flame lighted at all times. The pilot light may be blown out by wind or may go out because of low pressure in the line.

A distance of at least 10 feet is generally provided between waste gas burners and digestion tank covers or gas holders to prevent any possibility of igniting gas-air mixtures. Waste burners in chimneys are a hazard; instead waste burners should be located in the open for ready observation. An all-weather pilot flame for waste burners helps to ensure continuous burning of waste gases as they are discharged. Provision for condensate removal, pressure control, and flame protection ahead of waste burners is always required.

SCRUBBERS

In some installations, particularly where sludge-gas engines are used, gas purification needs consideration. Hydrogen sulfide in excess of about 50 grains per 100 cu. ft. in the presence of moisture (condensate) is rapidly corrosive to gas engines. Gas meters have been rendered useless by hydrogen sulfide after a few months of service at some plants. Hydrogen sulfide is also detrimental to lubricating oils used in gas-engine operation.

Hydrogen sulfide may be removed with a dry gas scrubber containing ferric oxide mixed with hardwood shavings.

Wet-type bubbling scrubbers, using plant effluent, may also be utilized for removal of hydrogen sulfide.

GAS UTILIZATION

Heating with sludge gas is similar to heating with natural or commercial gas. Equipment is nearly identical, except for modification of gas ports and air supply to secure proper combustion.

DIGESTER HEATING

The greatest use of sludge gas today is in the heating of digestion tank contents. There are two common methods of digester heating.

Hot water circulated in horizontal pipe coils within the digestion tank was first used for the heating of sludge. One major disadvantage in the use of internal horizontal heating coils is lack of opportunity to know the condition of the coils, as the digestion tanks must be emptied for inspecting the coils.

The most recent method of digester heating has been the use of external heat exchangers. This method of heating is particularly adapted to two-stage digestion where one digester is available for supernatant separation. The stirring of the sludge required for the operation of external heaters for single digesters poses the problem of securing a supernatant low enough in solids for return to the plant influent.

The external heaters may be of the countercurrent type in which the sludge moves through coils while the heating medium adjacent to the coils flows in opposite direction. Another type of unit is a self-contained heater and heat exchanger.

INTERNAL COMBUSTION ENGINES

Using the gas as fuel in internal combustion engines is often practical. An engine may be directly connected to a piece of equipment such as a blower or pump, and it is also common to connect one to a generator producing power which may be utilized in plant operation. The water used for cooling engines is often circulated through heat exchangers or heating coils in digestion tanks, thus serving a dual function of cooling the engine and heating the sludge.

DRYING AND INCINERATION OF SLUDGE

Drying of sewage sludge by means of digester gas is practised in a number of plants where sufficient excess gas is available. Heat from the gas is used to evaporate the excess water, usually from sludge which has been partly dewatered on vacuum filters. When sewage sludge is dried to 40 to 50% moisture content, the sludge itself usually has sufficient latent heat to support its own combustion. Incineration of screenings with sludge gas has been practised in several plants with specially designed incinerators.

TERTIARY TREATMENT

by

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An Address To
The Ontario Water Resources Commission
Intermediate Sewage Works Operators' Course
Toronto, Ontario
March 6, 1962

TERTIARY TREATMENT

by

W. A. STEGGLES

District Engineer

INTRODUCTION

Pollution Trends Today

Concentrations of population extending inland from the major water ways have created serious pollution problems, principally from the lack of dilution water. Even streams which once carried steady year round flows have diminished in size and constancy of flow, following urbanization, and have presented the sewage works operator with increasingly adverse receiving stream conditions with which he must cope. In many areas this problem is being solved by the construction of improved sewage works--notably in the Grand River watershed area. However, there remain other inland areas which cannot rely on large water-courses to assimilate sewage and trade effluents, and as long as bed-room type communities continue to establish and grow in these areas, the problem becomes intensified. We can look to little relief in the matter for the future, as the planners suggest to us that these inland areas will continue to grow at considerable rates.

Professor Gordon Fair (1) tells us that we made a fateful decision early in the 19th century when we permitted water carried sewage and trade wastes systems to become established as acceptable practice. In time, he sees the ultimate biological death or destruction of polluted lakes which cannot be relieved of accumulations of sludge on the lake bottoms, and anticipates the gradual tendency for a body of water to be progressively degraded by the continued addition of sewage nutrients.

What about our water resources--some authorities [2] tell us that things will get worse. Others [3] tell us not to worry--that those who cry doom concerning our future water needs have not really examined the facts. There is a "hidden" resource--that water which is not actually consumed--that is, it is used but not consumed. You and I are faced with the reclamation of these used waters. In this lecture, we will discuss more of the qualitative aspects than the quantitative problems which

confront conservationists. Mr. Hollis, U.S.P.H.S. tells us that in America by 1980, three quarters of the population will be urban and that reuse up to six times may become common.

And what about our yardsticks of pollution?--

There are people who caution against using only the BOD and suspended solids tests to measure and identify modern pollution--yes, modern pollution with all its subtle chemical characteristics which industrial development has spawned--such additional indicators as odour, fish taste and toxicity, and effects on water treatment processes and irrigated crops must be known. In addition, the author, Mr. Powers, (4) suggests that the following should be known as well--the concentrations of dissolved organic and inorganic matter, colour and turbidity. All these characteristics become very significant when waste treatment measures exceeding the capacity of secondary biological sewage treatment are considered.

After this brief and I hope alarming picture of water pollution today, you must be asking--but what about tertiary sewage treatment? Before we become involved in this however, let us consider some more background.

SIGNIFICANCE AND NEED FOR TERTIARY TREATMENT

Rather than enumerate the important reasons for treating wastes beyond the requirements of secondary treatment, I trust that the significance of these will become apparent as the lecture proceeds.

Along the train of thought outlined concerning the subtleties of industrial wastes, some further comment--the effectiveness of chlorine as a disinfectant concerning its ability to kill pathogenic and other bacteria is known. As well as the areas of doubt which have been created by various researches into its ability to kill viral entities, it is well known that simple chlorination is ineffective against many odour producing chemical wastes. Thus, we have such situations along the Lake Ontario waterfront and elsewhere where chemical wastes are drawn into municipal intakes and produce odours. Where these conditions develop, there are available modifications of the chlorine disinfection process which provide effective removal of these taste formers. It is suggested that the quantities of phenols and oils which we now accept as suitable for discharge to streams may soon require some form of tertiary treatment to reduce these pollutants to lesser levels--The Cities Service treatment plant at Bronte provides an example of this. Following elaborate secondary treatment, use is made of ozone and activated carbon to remove completely phenolic contaminants.

The role of synthetic detergents has been described by others in this course. Let it suffice to say that the presence of phosphorus derived from these sources provides an environmental factor in streams and lakes, which assists in the explosive production or blooms of these organisms which occur from time to time.

CLASSIFICATION OF STREAMS

Today, different provincial and state authorities use two methods in establishing the sanitary classification of streams. Both are arbitrary in nature and have implications for our subject.

In New England seven states have formed a water pollution control commission (6) and classified waters according to use. This classification was designed to reconcile conflicts of water use and is based on accepted water quality standards.

Class A - Suitable for any water use. Character uniformly excellent.

Class B - Suitable for bathing and recreation, irrigation and agricultural uses; good fish habitat; good aesthetic value. Acceptable for public water supply with filtration and disinfection.

Class C - Suitable for recreational boating, irrigation of crops not used for consumption without cooking; habitat for wildlife and common food and game fishes indigenous to the region.

Class D - Suitable for transportation of sewage and industrial wastes without nuisance, and for power. Navigation and other industrial uses.

Class E - Unsatisfactory waters falling below these descriptions, represent nuisance conditions.

Such a classification will require varying degrees of treatment, often variable throughout a given watershed. Obviously, many of the higher use classifications would necessitate secondary treatment plus or tertiary treatment.

Other areas have adopted another approach--a rational method which deals with the use requirements of each stream on its own merits. This method uses the following basic procedure in dealing with a particular stream: (7)

(a) The water uses are determined.

(b) Water quality criteria or requirements are applied to determine the suitability of the water.

(c) Waste control regulations compatable with engineering and economics are formed.

This is similar to the approach taken in this province and essentially, is characterized by the absence of a set of rigid standards.

BIOLOGICAL ASPECTS

Consider the case of a stream polluted by a sewage outfall--Immediately below the outfall, the aerobic bacteria degrade the wastes, metabolize and grow, creating a high demand for oxygen (ie. a high BOD). Subsequently, these organisms are replaced by anaerobic organisms which degrade the wastes yet further, producing alcohols, methane, hydrogen sulphide and ammonia. Added dilution and oxygenation permits the return of dissolved oxygen and the restoration of aerobic conditions whereby these chemicals are oxidized to carbon dioxide, water, sulphates, nitrates and phosphates. Thereafter algae growths are encouraged by favourable nutrient balance and frequently the algae themselves will give rise to secondary pollution.

Sewage treatment devices accomplish the same feat only under much more favourable environmental conditions. In the case of a lagoon or sewage oxidation pond, these processes are combined into one process unit in the presence of luxuriant growths of algae. The latter, through photosynthesis produce oxygen which is necessary for the heterotrophic organisms to assimilate the organic food matter present in the raw wastes.

Due to the abundance of nutrients in sewage effluents it has been thought that the luxuriant blooms of algae and hence the pollution of the stream could be eliminated by promoting this activity in lagoons. That is, plant effluent is introduced into ponds to promote algae growth and hence reduce the nutrient content of effluents.

REMOVAL OF NUTRIENTS

Domestic sewages contain significant nutrients which, as has been noted, promote growths in receiving ditches and watercourses. The nutrients can be reduced considerably in sewage oxidation ponds. Removals of nutrients in oxidation ponds are reported as follows: (11)

ammonia nitrogen - 75 to 90 percent reduction
organic nitrogen - 60 " "
phosphorus - 96 " "

As yet, nutrient removal from mechanical sewage plant effluents is not widely practised. Early work by the Department of Health found the removal of phosphorus by precipitation at high pH values from the Lindsay plant effluent to be expensive.

Work by Reid and Assenzo (12) has shown that the greatest removal of nutrients in oxidation ponds occurs where the densest populations of algae exist and in ponds which received treated sewage effluents. Relatively lesser production of algae was noted in raw sewage ponds. Regardless of whether the nitrogen occurs as ammonia, nitrites or nitrates, the algae will assimilate it, and providing there is a sufficiency of other nutrients, flourish. There is evidence that algae will as well assimilate organic wastes as well in a stabilization pond (19). These studies confirmed that:

(1) The organic matter produced by the algae was greater than in the original effluent, necessitating harvesting of the algae.

(2) Relatively large areas were required to effect substantial removals of nutrient from treated sewage effluents. e.g. At a flow of 1 MGD, it was estimated that 6 acres of pond are required to remove 2/3 of the phosphate, using a retention of 5 hours. (These results are based on the attached algae forms and not the free-floating algae. It was concluded that the former were less efficient than the latter as nutrient removers).

EXAMPLES OF TERTIARY TREATMENT

In the sense that tertiary treatment is used as a conservational measure whereby waste waters can be reclaimed for reuse, the following are cited as examples:

(1) Israel: Sewage effluent sprayed on sand dunes or discharged into depressions to permit percolation and thence recharge of ground waters. (13)

In Israel, a semi-arid country where water supplies are scarce, sewage is viewed in certain areas as a supplemental source of water. In an area to the south of Tel Aviv where ground water resources have been depleted this method is used on an experimental basis.

(2) Sweden: Compressed Air Revives Polluted Swedish Lakes. (14)

In a broader sense, this could be considered a tertiary sort of treatment--although somewhat belated.

In 1959, engineers placed a perforated plastic hose on the bottom of a grossly polluted lake and by aeration increased the oxygen content of the waters by 57 percent.

(3) Industrial Use of Municipal Sewage (15) (16)

Reuse of domestic sewage in Southern California (the effluent from a primary sedimentation plant) for irrigation of certain crops has been successful.

Large industrial plants in the United States are using sewage treatment plant effluents as economical sources of water for various purposes. One plant secures about 90 percent of its total water requirement from the use of 75 to 118 MGD of sewage effluent for processing and cooling use. Other uses include power plant cooling, boiler and process cooling waters. Water conservation, economy and lack of other more adequate sources of supply are usual factors in decisions to use sewage effluents.

When you think about it, our watercourses have been providing tertiary treatment ever since the early 19th century. Some of the devices and applications of effluent polishing processes will be outlined at this time.

(a) <u>Underdrained tile beds</u> are used to polish by filtration sewage effluents from secondary mechanical treatment devices. Example--Highway 400 Shell Gasoline station, Ontario.

(b) <u>Effluent sand Filters</u>--These devices are more widely used in Ontario as polishing elements following secondary sewage treatment. Often the effluent is chlorinated as well.

Examples--Stoney Creek - Hamilton area
Urbandale - Ottawa area
Westminster Twp. - London area
Toronto Twp. - Dixie and Erindale
Stouffville -

In these examples, the filters follow activated sludge plants or some modification of this process.

(c) <u>Micro-Straining of Secondary Effluents</u>

During 1959 and 1960, the Commission studied the use of a pilot plant micro-strainer in polishing sewage effluents at

Streetsville and Brampton. It was found, using an MKO fabric (the finest available) that reductions of 50 percent at least are obtainable in both BOD and suspended solids. This work provided more favourable results than previous studies made in 1956 at the Luton sewage works, England, where 38 and 54 percent reductions were obtained in BOD and suspended solids respectively.

Average results at Brampton provided a suspended solids content of 20 ppm. At influent levels of 20 ppm 50 percent or greater reductions were obtained.

These systems have proven effective in reducing the organic and solids material remaining in effluents from secondary sewage treatment plants. You are reminded that none of these devices actually achieve reductions of the nutrients contained in sewage, except the small quantities that are retained in sludges. These of course are carried into the stream and may result - through subsequent algae growths - in the secondary pollution referred to earlier.

Now, let us glance finally at the means available for excluding sewage effluents completely from watercourses and ditches. In the previous Israel example, this was achieved as you will recall for the purpose of water conservation and not as a stream protection measure - by percolation into the ground from a ponded natural depression.

Where the danger of ground water contamination is not significant this method is quite feasible.

Another method - irrigation - either broad or sprayed onto the land.

Recently in the State of Washington, a spray system was installed following a secondary treatment plant when the state required that no effluent would be tolerated in the stream at any time during the year. Likewise effluents are used for irrigating golf courses.

A proposal has been set before this Commission for the development of a residential subdivision above a conservation area where recreational bathing will be permitted. One of the alternative solutions under consideration is spray irrigation of the effluent from the proposed secondary plant.

SUMMARY

This lecture, I trust, has outlined the various practices

in use today whereby supplemental treatment is provided secondary sewage effluents. One of the most significant things to remember--an apparent high reduction of BOD in conventional treatment processes does not mean that the problem of pollution has been solved. Chemical wastes and nutrients often remain--and the latter in a form that is more readily assimilated by algae and other aquatic life.

Our problem today--improving technology whereby these adverse downstream conditions can be corrected economically.

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MODIFICATIONS OF THE

ACTIVATED SLUDGE PROCESS

by

G. H. KAY

District Engineer

An Address To
The Ontario Water Resources Commission
Intermediate Sewage Works Operators' Course
Toronto, Ontario
March 6, 1962

MODIFICATIONS OF THE ACTIVATED SLUDGE PROCESS

by

G. H. KAY

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INTRODUCTION

Although the activated sludge process has proven itself to be adequate in many installations, some locations at some times have experienced disappointing results due to many causes such as, excessive peaking of the sewage flow, hydraulic and organic overloading, changing sewage composition including toxic materials, unbalanced nutrients, inadequate aeration capacities etc. These often manifest themselves in bulking sludge conditions in the final clarifier, with associated effluent quality deterioration.

Some limitations of the process are considered to be:

1. High initial oxygen demand of the mixed liquor.
2. High air requirements.
3. High solids loading on final clarifiers.
4. Tendency to produce bulking sludge.
5. High sludge recirculation ratios required for high B.O.D. wastes.
6. B.O.D. loadings limited to about 35 lb/1000 cu. ft. This requires relatively long retention times, yielding high capital investments.
7. Inability to produce an intermediate quality of effluent.

To overcome these problems and to continue to improve the basic process, much experimentation has been accomplished, particularly by plant operators, and innumerable modifications have evolved. It is not the intent to catalogue all these in this lecture, but only to discuss some of the more widely accepted modifications of the conventional process.

GENERAL

The activated sludge process and its modifications have time-solids relations or stated otherwise, within limits, if the active solids in the process are increased, the time

required for purification is reduced proportionately, and vice versa. To reduce the size and therefore the cost of capital expenditures for plants, the tendency has been to carry as high a concentration of aerator solids as possible. This procedure has limitations due to present-day oxygenation and sedimentation processes. Temperature also affects the process but generally this aspect has not received as extensive attention to date.

The oxygen demand of the mass of return activated sludge is at a relatively low level, but as the fresh colloidal, dissolved, and finely divided solids of the sewage is attached to the floc, biological activity is stimulated and the oxygen demand thereof is immediately dramatically increased. The inlet demands can approximate 4-5 times those at the outlet end.

It is difficult to obtain a true value of dissolved oxygen values at the inlet and for this reason, these values are determined at the outlet end where adequate values are maintained to avoid zero concentrations at the inlet end.

TAPERED AERATION

Since the diffused aeration devices would ordinarily be spaced for equal supply, D.O. concentrations could become zero at the inlet end of the tank. This would result in the activity of the bacteria being retarded. A retardation of bacterial activity would necessitate longer aeration times and thereby larger units, or poor, (high) S.V.I. values, settling characteristics and effluent qualities will occur. To combat this, aeration devices are located sometimes in varied concentrations along the aeration tank length, with the maximum concentrations at the inlet end. This "modification" is termed Tapered Aeration. This allows the oxygen supply to approximate more closely the oxygen demand to avoid retardation of the bacteria's activity, yielding better treatment results and lower aeration costs.

STEP AERATION

This process has the sewage added to the return activated sludge rather than the reverse, as in the conventional process. This is done at many points on the combined length of the aeration tank. This too allows the initial high oxygen demand to be more evenly distributed. Around-the-end baffles are usually required to avoid short circuiting.

The name "step aeration" suggests that the process is set up only for a purpose similar to tapered aeration. Its other attributes will be discussed further, later in this paper.

As stated above, the conventional and modified processes have time-solids concepts. Basically this suggests that if proper F/M ratios are maintained, greater concentrations of solids

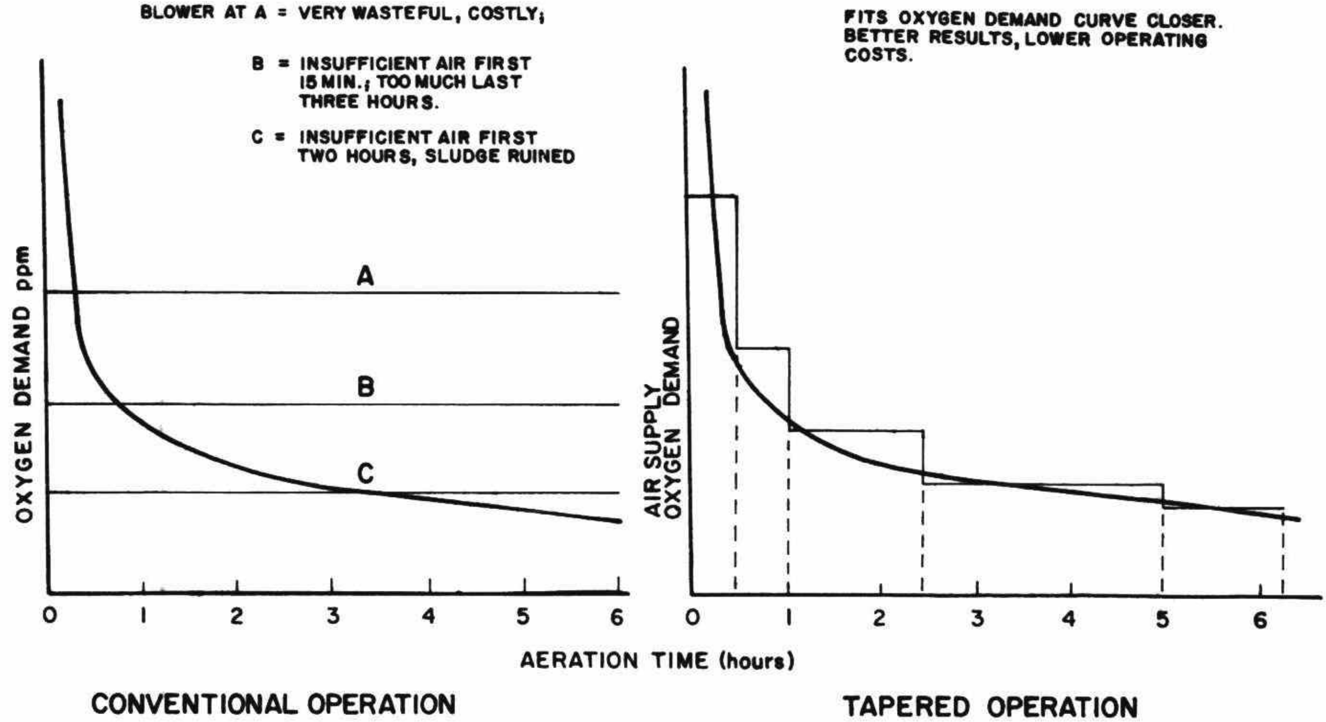
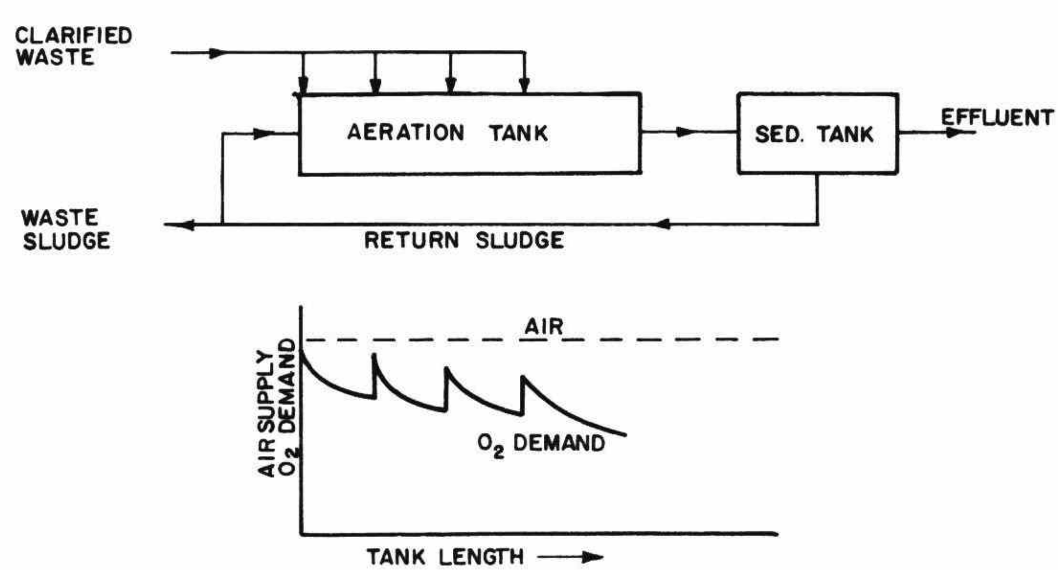


FIG. 1 TAPERED VS. CONVENTIONAL AERATION



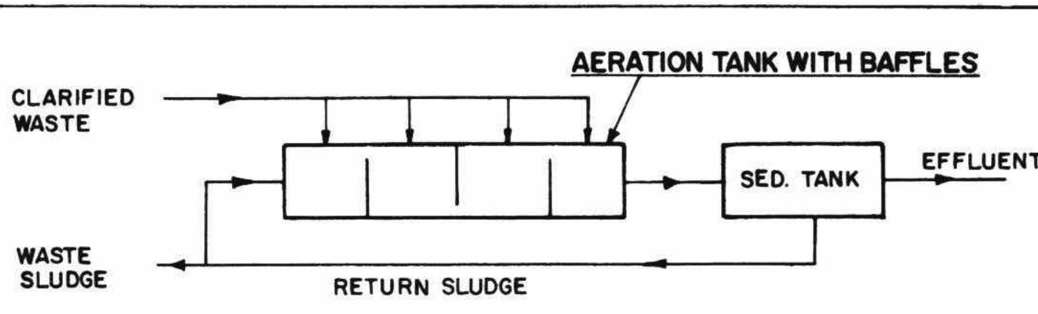


FIG. 2 STEP AERATION

will result in lower required total aeration times. This is also limited somewhat by the settling tank capacities. In the step aeration modification, adding sewage in increments allows a return sludge with higher solids concentration since these will be reduced to acceptable limits by dilution, before discharge to the settling tank.

DISCUSSION OF THE ACTIVATED SLUDGE PROCESS AND ITS UNITS.

The basic unit of the activated sludge process is an aeration vessel. Primary settling tanks allow the removal of the larger solids of the sewage that can be easily removed by sedimentation for disposal elsewhere, and so lower the load imposed on the secondary units that are necessary for conversion and removal of the non-settleable material. In this lecture, we are concerning ourselves with the secondary treatment units. Suffice to say that secondary units may or may not be preceded by primary units, and the load on the secondary units will reflect this.

The aeration tank provides the aerobic environment for adequate acclimatized sludge added to the sewage, to rapidly clarify it by adsorbing the suspended and colloidal organic material and by absorbing the dissolved nutrients thereof.

This desired result, which often can be completed in approximately one half hour, would be difficult and more expensive by any other means. The clarified liquid portion now usually containing less than 10% of the raw sewage's B.O.D. and suspended solids contents, could be discharged to a watercourse.

The sludge in its present state now contains, the solids removed from the sewage, in or on the surface of the active organisms in its zoogleal mass. This material, together with the cell material of these organisms themselves, is highly putrescible. It is necessary therefore, to ensure that this material is not discharged to the receiving waters. To do this requires separation of these solids, from the liquid to be discharged.

If the aeration process would be interrupted and a quiescent period established in the aerator, the solids would settle, and the inoffensive liquid could be discharged as the aerator effluent.

Below the liquid, there would then be a putrescible mass on the bottom of the tank representing the original activated sludge mass plus the sewage solids.

If this is to be a continuous activated sludge process, a portion of this sludge must be mixed continually with the incoming sewage. However, these quiescent periods in the aeration tank to allow sludge separation, are not practicable. Therefore a separation, sedimentation, or settling tank is provided to complement the aeration tank. This allows the sludge

to be settled and returned for continuous mixing with the incoming sewage, without the need of aeration interruption to provide quiescent periods in the aeration tank.

If aeration is provided for only the thirty minutes required to provide clarification, the sludge with its absorbed and adsorbed sewage solids, is not in a condition to clarify the new incoming sewage and will require a conditioning period under aerobic conditions to improve its clarifying and settling potential. In the conventional process, this is achieved by providing this extra retention time in the aeration tank prior to any final sedimentation. Here the sludge undergoes biological oxidation for approximately six hours longer, in the liquids that will later constitute the plant effluent.

SLUDGE REAERATION

This aeration period however, is needed for the purpose of conditioning only the sludge, not the effluent liquids, and this fact was used in the development of some modifications of the process.

In these, a new tank for conditioning the sludge alone, usually called a sludge reaeration tank, is provided on the sludge return line between the final settling tank and the head of the original aeration tank. Here the sludge that has been in aerated contact with the sewage for 30 minutes and then collected in the final settling tank, (after separation by sedimentation from the future plant effluent), is aerated alone, often for the common six hours of the conventional process.

Since this sludge usually amounts to less than 40% of the total sewage flow, this tank can be substantially smaller than the original aeration tank. Also, as previously stated, the aeration time required to allow settling and clarification, is approximately only 30 minutes. Therefore, the size of the tank originally known as the aeration tank, is reduced appreciably to supply only this 30 minutes retention time and this tank is now known as the contact tank.

This modification was almost prophesied by Arden and Lockett in their 1914 Summary and Conclusions when they said that while oxidation of the solids in the sludge procedes, the sludge need not be in contact with the sewage.

Therefore, it can be seen that the required total aeration capacity, (in contact tank and sludge reaeration tank), can be reduced from the conventional process aeration capacity, while producing equivalent process efficiencies.

It should be noted here that the conventional continuous process plant consists of at least an aeration tank, final settling tank and an activated sludge return mechanism. This new modification called Sludge Reaeration, splits the aeration unit into two tanks, yielding three units usually called the contact tank, final settling tank and sludge reaeration tank, together with a sludge return mechanism. It

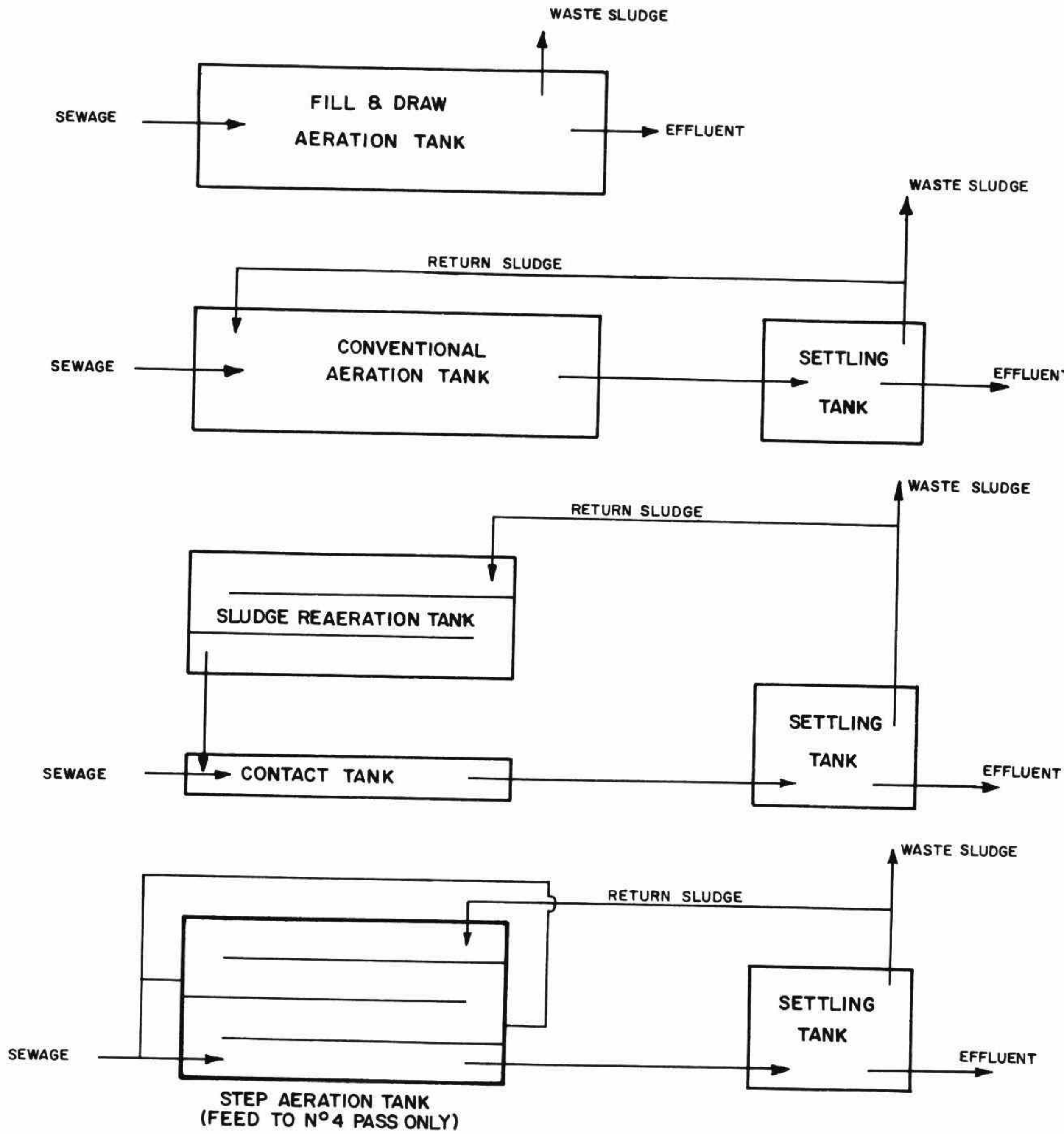


FIG. 3 SCHEMATIC DEVELOPMENT OF SLUDGE REAERATION MODIFICATION DEVELOPMENT, INCLUDING STEP AERATION.

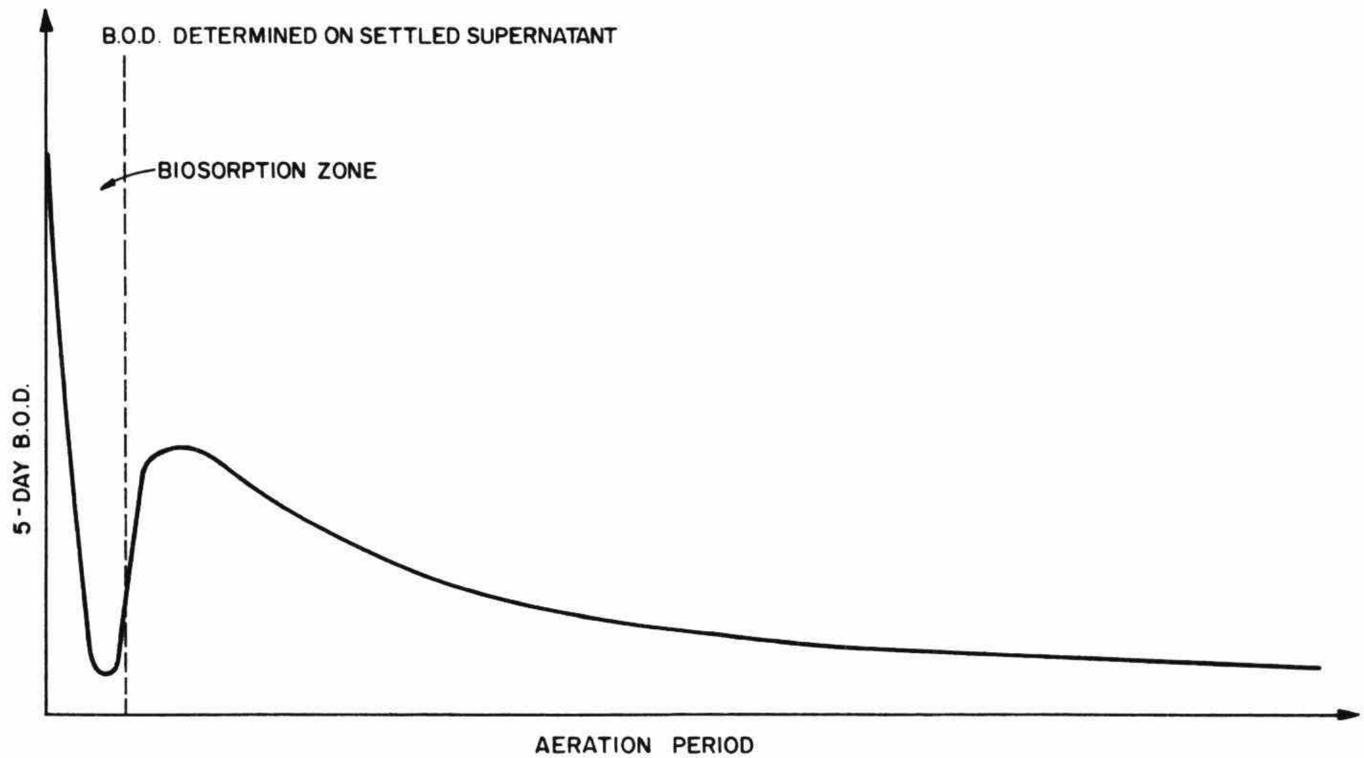


FIG. 3A VARIATION OF 5-DAY B.O.D OF RAW SEWAGE-ACTIVATED SLUDGE MIXTURE WITH AERATION PERIOD.

is considered by some, that this process can effect a saving of 20% in the total aeration unit capacity required.

Sludge reaeration usually permits the use of a lower solids concentration in the mixed liquor of the contact tank than would be required in the conventional plant, to produce the same degree of purification in the same period of time. Therefore, it also permits, lower rates of sludge return, lower settling tank loading and possibly higher settling tank efficiencies.

Times required usually vary from 30-90 minutes in the contact tank, amd 5-7 hours in the sludge reaeration tank. Large quantities of active solids are maintained in the reaeration tank, out of contact with the changing incoming sewage, and therefore the process is considered to be less subject to shock loadings.

Prior to 1938, approximately only 10% of the process' total aeration capacity was utilized for sludge reaeration. Since that time, new flow diagrams have yielded reaeration percentages of total aeration capacities in the ranges of 50-67, 80-86, and 68-70, and so allowed greater acceptance of the modification.

BIOSORPTION PROCESS

Whereas the Biosorption process may not have been the first major application of the above modification, the work by Ullrich and Smith at the Austin, Texas Sewage treatment plant gave a well published insight into the possibilities thereof.

As with many developments, this modification was necessitated by operating problems at the existing plant. The sewage at Austin was always difficult to treat, with frequent sludge bulking, probably due to periodic overaeration, so that during the period 1948-1951, experiments were performed, resulting in complete plant conversion to Biosorption, (extended reaeration) procedures, in 1954. Here, basically by increasing only the number of clarifiers, and converting the aeration tank to suit sludge reaeration techniques, the capacity of the plant was increased from 6 to 14-16 M.G.D.

At this plant there are no primary settling facilities. In most applications of the sludge reaeration modifcation however, these units are usually included.

In this modification, activated sludge approximating 40% of the total flow, that has been aerated or stabilized for 5-7 hours, is added to the sewage in the contact tank. Contact time is 20-40 minutes. Clarifier retention time is usually 1-3 hours. Waste sludge is withdrawn from the final clarifier or from the aerator. Since this sludge contains a high energy or food level due to a high volatile solids content, a dispersed or fluffy growth yielding waste sludge problems, would be anticipated. In fact, the sludge does not compact readily and thickening units are required to obtain concentrations of 2.5%

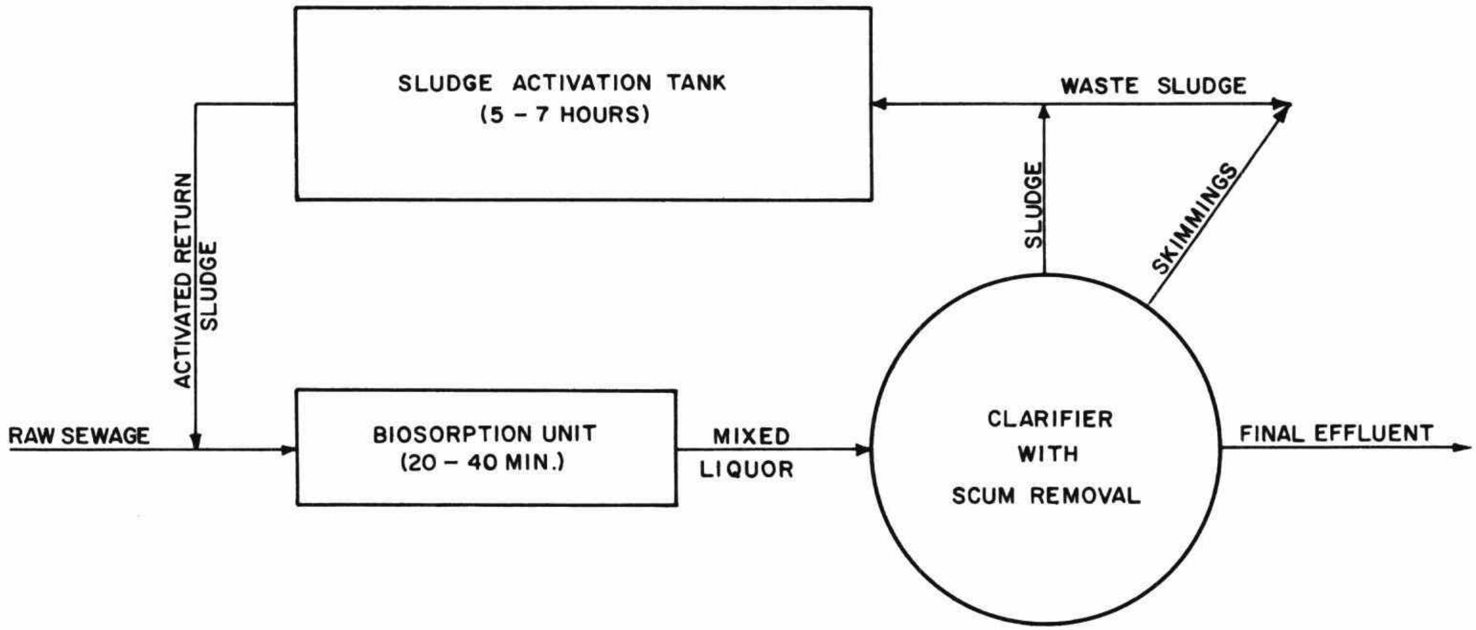


FIG. 4 SCHEMATIC DIAGRAM OF BIOSORPTION PROCESS

solids. This poses unusual problems in sludge disposal and lagooning of sludge is practised at Austin. Some newer application of this process have utilized other methods of sludge disposal including aerobic digestion.

However, it is claimed by Ullrich and Smith, that the process is less tempermental, less susceptible to shock loads and requires approximately only one half of the usual aeration capacity.

THE KRAUS PROCESS

L. S. Kraus, chemist of the Greater Peoria Sanitary District, evolved a modification using sludge reaeration with further refinements. The Peoria plant had been designed as a conventional activated sludge plant but bulking of the sludge regularly produced poor effluent qualities. This was in part due to shock loads which approached values of three times design flow and these contained high-strength packing house wastes.

It would appear that these variances upset even the new sludge reaeration modification here, by yielding continued bulking in the final settling tank.

As we have discussed, a low S.V.I. or a high S.D.I. usually indicates a sludge having good settling characteristics. Therefore, since the S.V.I. is equal to

$$\frac{\text{30 Min. settling test (\%) x 10,000}}{\text{M.L.S.S. (p.p.m.)}}$$

Kraus chose to add some heavy solids to the aeration units to increase the M.L.S.S. content and possibly reduce the 30 min. settling test values, to lower the S.V.I. values, and so reduce the incidence of bulking. He did this by adding digested sludge and digester supernatant to the sludge reaeration tank, or as Kraus called it, the nitrifying aeration tank, to provide these heavy solids. The induced settling action of these additives could have been produced by adding other inert material such as clay, diatomaceous earth etc. However, this tank is probably the best location to discharge the supernatant and it is felt that these highly nitrified solids tend to reduce the upset caused by shock loads, in part by providing nitrates to aid in satisfying the high initial oxygen demand. Also the aerated digester solids are considered to be converted rapidly to a heavy activated sludge.

Kraus was required to initiate a different aeration technique to provide the increased oxygenation required by the solids in the reaeration tank. This constituted two sets of aerators, being on the same or opposite sides of the tank, at different levels and so was termed Dual Aeration. This requires air to be supplied at two different pressures, due to the different depths of entry.

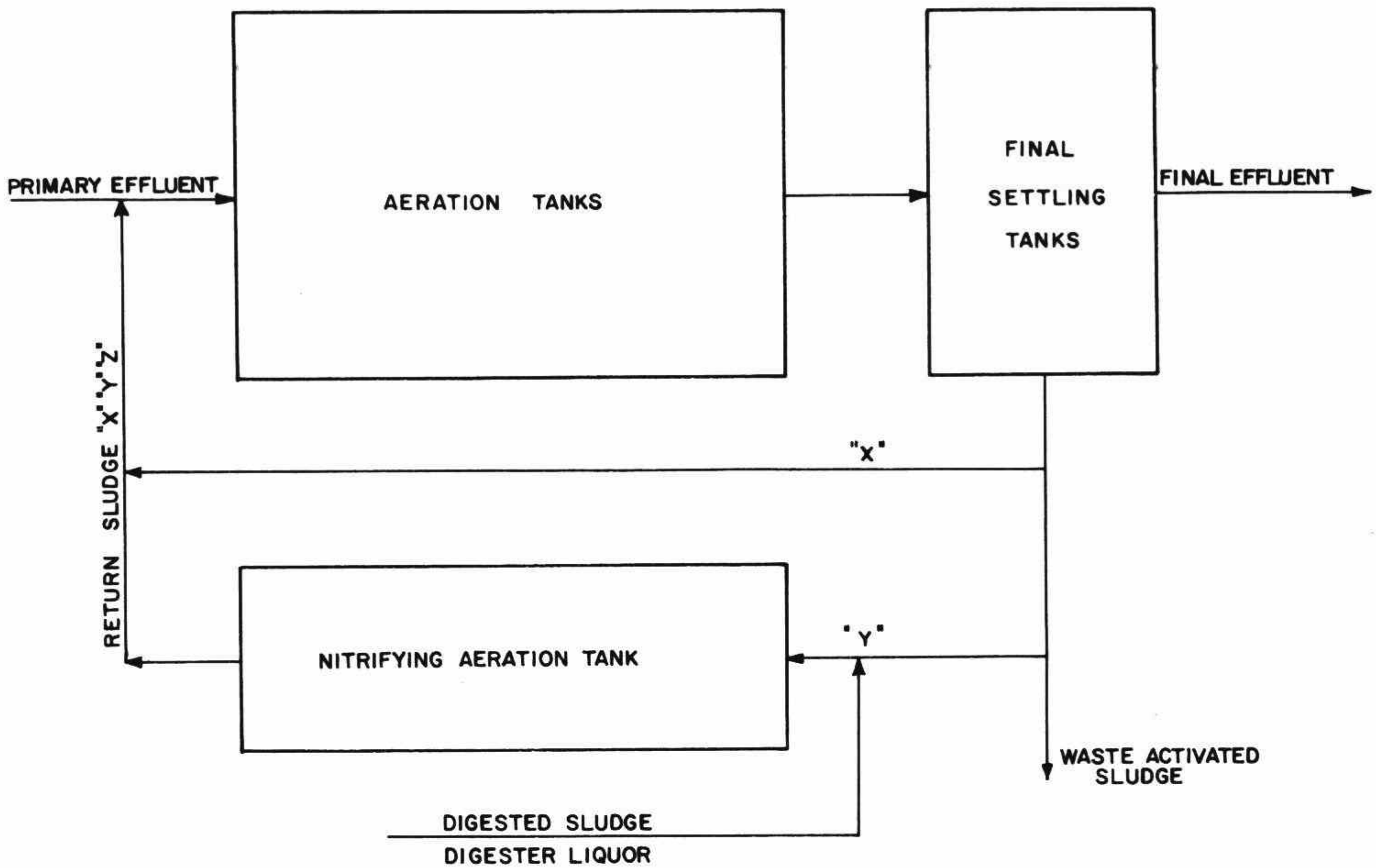


FIG. 5 FLOW DIAGRAM OF KRAUS MODIFICATION

STEP AERATION

This process is also called "Distributed Loading", "Multiple Port Dosing", or "Incremental Feeding."

These names probably describe the process in all its aspects more properly than the title "Step Aeration". As shown in drawing number six, the procedure is to cause the return sludge to flow through the aerator with incremental additions of sewage throughout its length. As well as leveling the oxygen demands, for equivalent average aerator solids concentrations, higher solids concentrations are possible in the initial passes for accelerated stabilizing action, with low concentrations entering the final settling tank due to dilution by the sewage. This tends to allow improved settling efficiencies.

Note also that the original pass has no sewage added and therefore acts as a sludge conditioning tank with high solids concentrations.

Consider the situation if all the sewage is added to the last pass or channel of the aeration tank. The system then becomes a true sludge reaeration system having contact tank to stabilization tank capacity ratios of 1 to 3 or 1 to 4 depending on the number of passes.

This process is also very flexible, a quality that is extremely desirable in any secondary type sewage treatment plant due to continually changing sewage loadings etc.

The regular sludge reaeration plant has its contact tank and sludge reaeration tank capacities fixed, so that at design flow, the retention times therein, are also fixed. Since the sewage flow rates or strength may change, during the day, seasonally and over the years, the retention times in these units may change and so become inadequate, with potential deterioration of process characteristics. Only the alterations provided the conventional process, such as sludge return rate alterations and thereby M.L.S.S. concentrations alterations, are available to overcome these difficulties.

However, in the Step Aeration process, with decreasing S.D.I. values, the point of addition of sewage can be changed to only the last pass or passes of the aerator. This decreases the contact period, decreases the amount of active solids going to the settling tank and therefore, with the same rate of sludge return, increases the solids carried in the reaeration portion of the tank. The increased reaeration time will allow the solids to be properly conditioned for later addition to the contact portion of the tank. Then as the S.D.I. increases, the process can be slowly reverted to ordinary sewage increment distribution to all passes or channels, and so a safety factor is provided once more, against a similar future plant upset.

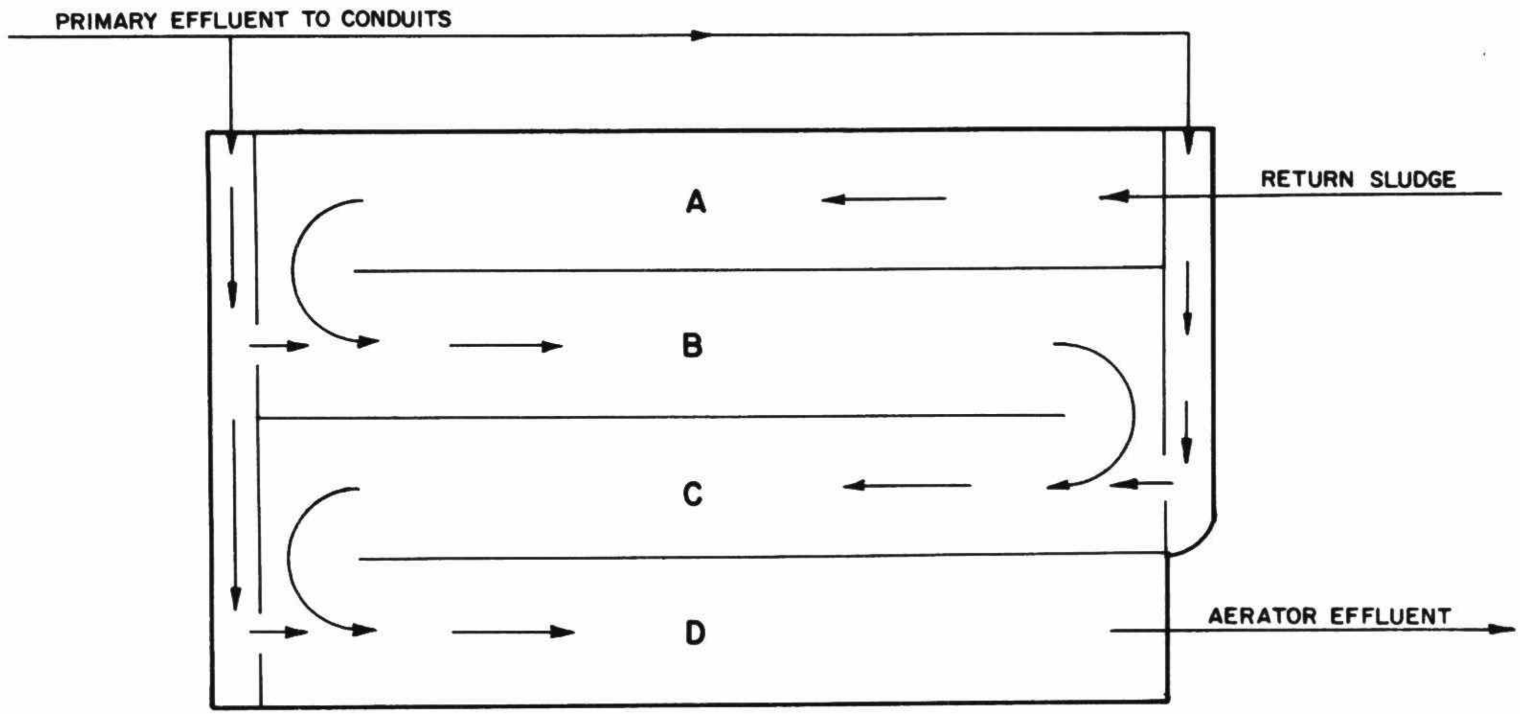


FIG. 6 STEP AERATION PROCESS EMPLOYING A FOUR PASS TANK WITH ADDITION OF SEWAGE IN B, C AND D PASSES.

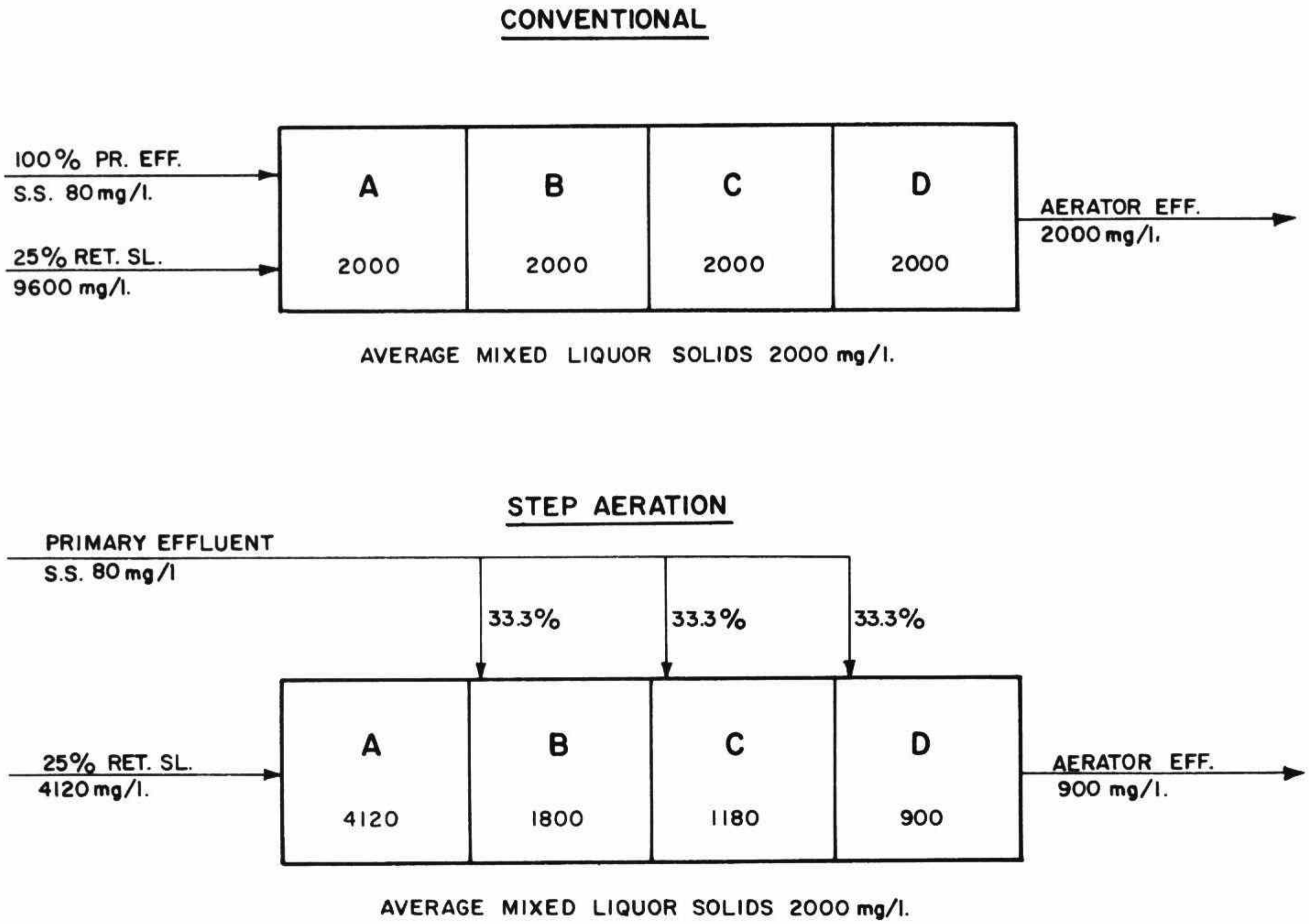


FIG. 7 COMPARISON OF SOLIDS IN RETURN SLUDGE AND AERATION TANK EFFLUENT FOR A GIVEN PLANT OPERATING ON CONVENTIONAL AND STEP AERATION SYSTEMS

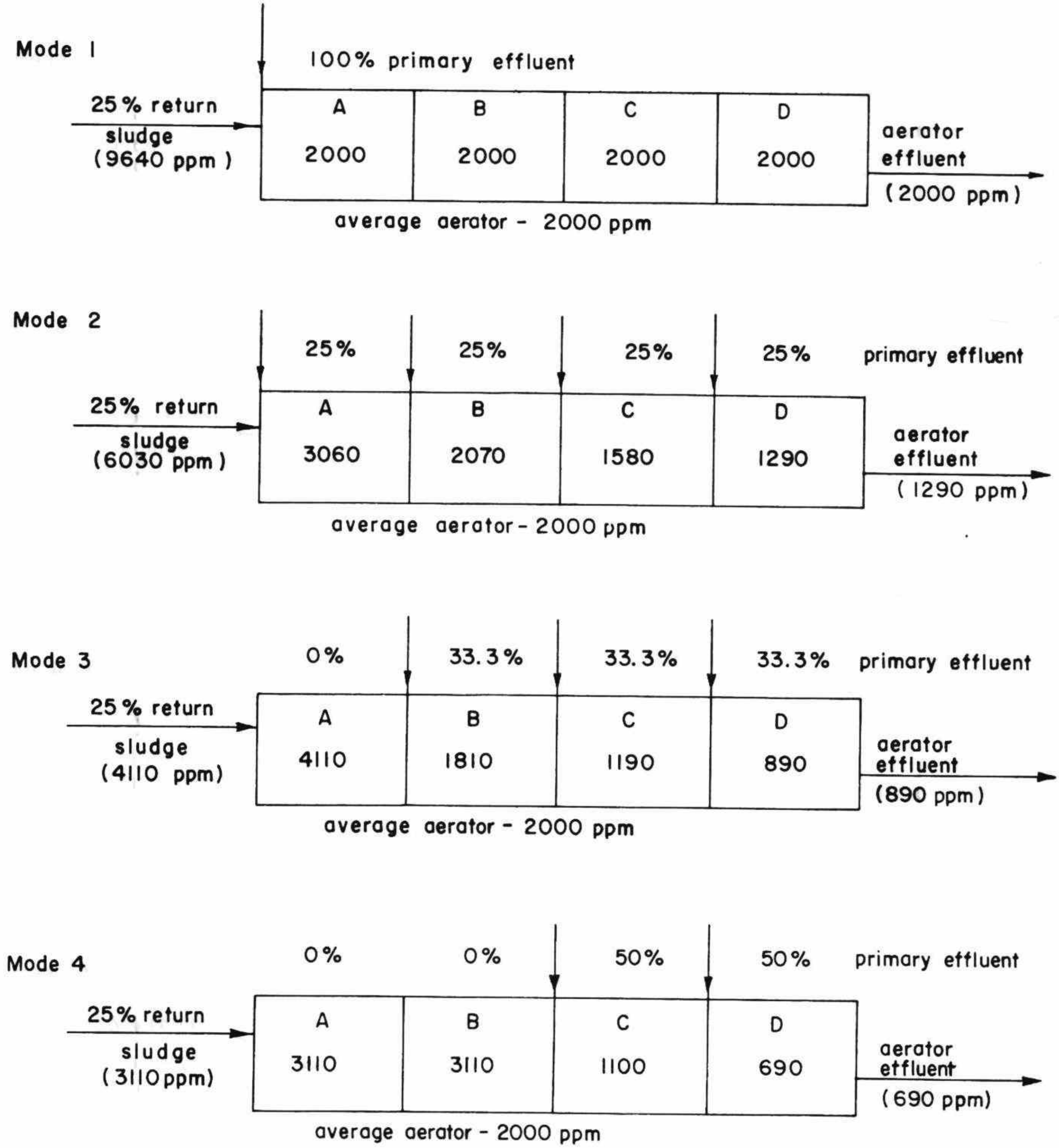


FIGURE 8

STEP AERATION

INFLUENCE OF DIFFERENT MODES OF ADDITION ON AERATOR CONCENTRATIONS AT CONSTANT AGE

In a conventional plant when the S.D.I. is decreasing, the sludge now being more voluminous, would occupy more space in the final settling tank. The required increased sludge return rates would lower the aerator solids concentration and the retention time in the aerator. We have said that the process has a time-solids relation; since both of these are being reduced, further aggravation of the sludge quality would occur. Conditions would continue to worsen until either the incoming sewage rate or strength would be reduced naturally or through by-passing.

The advantages of the step-aeration process then, are the ability:

1. To distribute the sewage load effectively throughout the aeration tank.

2. To regenerate the sludge more effectively by maintaining higher concentrations of solids in the inlet end than in the effluent end.

3. To circulate a relatively smaller amount of sludge solids throughout the final settling tanks.

The advantage of being able to vary the average concentration of suspended solids in the aerator and thus the sludge age or loading rate, by shifting the pointsof application of sewage, is very important, adding extreme flexibility to the operation of the plant. It allows the operation at a greater sludge age.

Loadings of 55-84 lb. of B.O.D./1000 cu. ft. of aerator are common: the conventional plant loading is 25-30 lb/1000 cu. ft.

It can be operated with no reaeration or with 25, 50 or 75% of tank volume being used for reaeration (in a 4-pass unit.)

Return rates are usually 28-34%. It is the intent to pass the least quantity of activated sludge to the final clarifier, while maintaining adequate sludge age and thereby a proper F/M ratio. Average aerator solids are usually double those of the conventional plant, so that aeration time becomes 2.4 - 4.7 hours and the aerator unit can thereby be smaller in size.

SUMMARY

The main Sludge Reaeration modifications are known as:

1. Extended Reaeration

2. The Hatfield Process

3. The Kraus Interchange Process

4. Step Aeration

wherein Extended Reaeration includes those processes known as Biosorption, Contact Stabilization and the Ridgewood Biological Coagulation Process.

Total Oxidation

In the conventional activated sludge process and most of its modifications, the average F/M ratios, using the volatile M.L.S.S. (V.S.S.) usually are kept at no less than approximately 0.3-0.35 and the Gould sludge age at no more than approximately 3.5-4.0 days, to maintain the sludge in an active state. That is, not allowing it to remain in the death phase, where endogenous respiration predominates, for excessive periods of time. This results in a conversion of sewage to sludge cell material approximating .5 lb. per lb. of 5 Day B.O.D. removed. The sludge in excess of the return sludge required, must be removed, to maintain these ratios. This is often treated in separate units being anaerobic or aerobic digesters, since the material is putrescible and cannot be otherwise easily stored.

In the total oxidation or extended aeration process, the F/M ratio is kept at such a low level (approximately 0.05 based on <u>volatile</u> M.L.S.S.), as to ensure that all the sludge spends a prolonged period in the death phase (endogenous respiration phase). This produces an older, less active sludge but a great number of organisms will be present. It also ensures that the sludge will be self-consuming to produce a minimum of stablized sludge for disposal either in the effluent or by periodic removal. Primary settling tanks are not commonly used. This then allows the anaerobic digester with its associated operating problems to be unnecessary and makes the process attractive to installations where these units are not desirable or adequate supervision is unattainable. The extended oxygen demand of the larger colony of organisms aerobically digesting these excess solids, increases aeration costs so that the plants are more commonly found in the smaller sizes.

Care is required in translating existing parameters for sludge age and F/M ratio from the conventional process, to the extended aeration modification. It becomes more critical to know the <u>active</u> microbial portion of the volatile content of the M.L.S.S. This may be only 10% of these volatile suspended solids (V.S.S.) and is used by some researchers in obtaining F/M ratios.

The retention time of the sludge at equilibrium as opposed to the Gould sludge age, is related to the fraction of the sludge wasted each day and can be calculated by dividing the M.L.S.S. by the quotient of the effluent suspended solids concentration divided by the total retention time of the raw wastes in the entire system. The retention period for an extended aeration system with 24 hours aeration and 4 hours sedimentation would be calculated on the basis of 28 hours total retention. If the effluent averaged 28 mg/l total suspended solids and the M.L.S.S. averaged 2000 mg/l total suspended solids,

for effluent samples from these plants due to the effect of nitrification reactions in association with the carbonaceous B.O.D. which may cause the processes' efficiency to appear lower.

AERATED LAGOONS

Sewage lagoons or oxidation ponds have been receiving a good deal of interest. Where acceptable land is economically available, these provide a unit having very low operating costs. The Commission has been using a design criteria of 1 acre per 100 persons, with liquid depths of 3-5 feet. Here algae and surface aeration, provide the required dissolved oxygen together with little associated mixing and long detention times. Recently, to reduce the area required for new installations or to avoid enlarging existing units, mechanical aeration units have been added to the lagoon. The lagoon then resembles somewhat, an activated sludge plant within an earthen pit. Eckenfelder has suggested that 4-7 days retention time is required. If a settling area is not provided, high effluent suspended solids will prevail.

Completely Mixed Systems

This classification is intended to contain all those activated sludge plants which employ aeration units wherein the influent sewage is quickly and intimately mixed into the entire aeration tank. The best known of this type is the so-called "package" type plant using a vertical draught tube. These plants had been accepted due to their structural convenience, for design flows less than 1 M.G.D.

If the microrganisms are to operate at maximum efficiency at all times it was felt that they should be maintained in a constant state of growth rather than the feed-starve cycles of the conventional plants. In such a state, the bacteria would always be adapted to the type and concentration of organic material in the raw waste. The aeration tank also acts as a surge tank.

Dispersal of the load throughout the aeration unit relieves shock conditions, equalizes oxygen demands, and provides for uniform operation. This feature makes the process attractive to smaller installations which experience intermittent shock loads.

These units may operate on a basis of high synthesis of sludge with scheduled wasting, or on a basis of "complete oxidation" employing endogenous respiration. The only differences are in the need for sludge wasting facilities in the former case, and greater air requirements in the latter.

SUMMARY

Under optimum conditions, activated sludge can make it possible to clarify domestic sewage in approximately one half hour. It is then necessary to condition this sludge and the sewage solids it has removed, by biochemical oxidation. This does not need to take place in contact with the liquids which will become the plant

the average sludge retention period would be calculated as follows:

$$t = \frac{2000}{\frac{28}{28}} = 2000 \text{ hrs.}$$

It can be seen that care is required in using conventional activated sludge operating parameters for evaluating extended aeration plants.

M.L.S.S. ranges are usually good - 6000 - 8000 p.p.m. and the process is capable of reductions of at least 80-90% B.O.D. and 70-80% suspended solids.

The plants are usually designed to provide 24 hours aeration rather than the usual six, with 4 hours settling capacity rather than the usual 2 of the conventional process. These values are chosen primarily to ensure adequate treatment of peak flows. The aeration time can be reduced by carrying higher M.L.S.S. concentrations, if oxygenation, settling, and sludge returning capacities can always be sufficient. In small plants, to avoid septicity or dentrification in the final tank with such high solids concentrations, sludge return rates of 100% have been usual, 200-300% are common and values of 400-500% may emerge. If dentrification occurs it will result in rising sludge conditions.

The high M.L.S.S. concentrations carried in this modification, usually ensures good B.O.D. reduction, but if severe solids build-up is allowed, in present day settling units, some of the solids tend to float out in the effluent to increase its suspended solids content. Ratios of B.O.D. to suspended solids of approximately four to one therein, are common. S.V.I. values generally range from 40 to 80.

It can be seen then that the settling tank is the key to acceptable effluents from this process, and dynamic removal such as centrifuging, rather than gravity-induced settling, may ultimately be required.

The title "Total Oxidation", is misleading since there is a portion of the bacterium's cellular capsule, apparently composed of polysacharides, which is resistant to decomposition and therefore, unless adequate sludge wastage is allowed and achieved in the effluent, periodic positive removal of solids is required. However, this is a relatively stable material and can be discharged to an open storage tank if necessary, for subsequent disposal. High quality effluents can be produced only with separate sludge wasting at periodic intervals. It is preferable to waste an appreciable percentage (20-30%, maximum 50%) of the M.L.S.S. at each wasting, to ensure proper removal of the inert material content.

Then, since there is not complete or total oxidation of the sewage solids, the name "Total Oxidation" is a misnomer. The term "Maximum Active Solids" (M.A.S.) or more preferably "Extended Aeration" would be more acceptable.

It may be required to revise the B.O.D. analysis

effluent, and several modifications utilizing separate aeration or reaeration of this sludge have evolved. These can provide improved operations and reduced capital costs.

Since the activated sludge process and its modifications also have time-solids relationships, providing a proper F/M relation is retained, the tendency is to aerate the sludge at high concentrations to reduce aeration times and unit sizes. This will require increased oxygenation refinements.

As the sludge must still be separated from the liquids of the sewage, improvements in settling tanks or dynamic separation mechanisms must keep pace.

To date, many modifications to the process have been introduced. The Step Aeration modification with its great flexibility and Extended Aeration (Total Oxidation), with its deletion of anaerobic digesters for smaller plants, appear to be the most widely accepted of these.

DISPOSAL OF SLUDGE

BY TRUCK HAULAGE

by

G. R. TREWIN

Supervisor, Sanitary Engineering

An Address To
The Ontario Water Resources Commission
Intermediate Sewage Works Operators' Course
Toronto, Ontario
March 7, 1962

DISPOSAL OF SLUDGE BY TRUCK HAULAGE

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G. R. TREWIN

Supervisor, Sanitary Engineering

INTRODUCTION

The paper "Sludge Handling Methods", presented at the first operators' course, dealt with common methods of sludge disposal. Here consideration will be given to one particular segment of this topic.

The lecture will cover the disposal by truck haulage of both raw and digested sludge in the liquid or filter cake form. Two pertinent factors, public health considerations, and fertilizer value of sludge will be included in this review. The main subject will be covered under the headings:

Sludge Quantities

Vehicle Design

Public Relations

Disposal Programme

PUBLIC HEALTH CONSIDERATIONS

Many conflicting reports have been made regarding the sanitary status of raw and digested sludges. In the Manual of Practise No. 2 issued by the W.P.C. Federation consideration is given on the survival of the listed organisms under various conditions.

Typhoid fever bacili

Vibrio cholerae

Mycobacterium tuberculosis

Virus of poliomyelitis

Entamoeba hystolytica

Other parasites

It is generally agreed that heat dried sludges may be used for soil conditioning purposes with confidence from a sanitary standpoint. Only one Ontario plant can produce such a product.

Many variables affect the survival and transmission of disease causing organisms in the other forms of sludge. A few of the variables are: drying time, digestion, storage period, temperature, and pH. Adequate digestion is said to be effective in eliminating most of the typhoid fever bacili. It is considered safer to use digested sludges for soil conditioning purposes.

Both raw and digested sludges may be spread on land where adequate aging and cultivation is effected. Where an adequate aging period is not allowed the land should not be used for crops which may be eaten raw. At no time should either raw or digested sludge be spread on growing crops which may be consumed raw.

Unless sludges are effectively heat dried they should not be spread on active grazing land. Forage crops which have been treated with sludges not rendered innocuous by heat drying should be cured before use.

The following literature may be consulted for further information:

(1) U.S. Dept. of A. Circular No. 972
Sewage Sludge for Soil Improvement

(2) Manual of Practise No. 2
Federation of Sewage Works Assoc.
Utilization of Sewage Sludge as Fertilizer.

FERTILIZER VALUE

With the exception of the nitrogen and phosphorus content in undigested activated sludge, the fertilizer content in sludge is small. Therefore, the greatest percentage of sludge products are classified as soil conditioners and not fertilizers. Nevertheless, this material, whether termed as fertilizer or soil conditioner, can provide valuable humus and trace elements to the soil. The three main constituents required in commercial fertilizer are nitrogen, phosphorus and potassium. Nitrogen and phosphorus are available in good percentage in sewage sludge while potassium is generally present in amounts less than one percent.

The type of treatment from which the sludge originates as well as the nature of the raw sewage, have a great bearing on the value the resultant sludge as a soil conditioner. The following table presents analysis results from various types of sludge.

Type of sludge,	City	Primary fertilizing constituents as percent		
		Nitrogen N total	Phosphoric oxide P205 total	Potassium oxide K2O
Activated sludges				
	Chicago	4.81	6.86	0.30
	"	5.60	6.97	.56
	Houston	5.77	3.08	.30
	McKeesport	5.68	7.38	.61
	Milwaukee	5.96	3.96	.41
Digested activated sludge				
	Des Moines	1.81	3.31	.40
	Hagerstown	3.13	2.81	.10
	"	4.71	4.96	.74
	Los Angeles	2.49	4.07	.21
Digested sludges, primary treatment				
	Beltsville	1.89	1.64	.19
	Greenbelt	3.12	.91	.24
	Washington	2.06	1.44	.14

The above data indicate the relative value of the various types of sludges. In order of value the undigested activated sludge is first with digested activated sludge, raw primary sludge, and digested primary sludge following in the given order. One draw back in evaluating the nitrogen content is that not all of the measured nitrogen is readily available for plant use. This factor reduces its commercial value but on the other hand it creates a residual fertilizer effect and also prevents the burning of lawns and plants.

Tests in San Diego have revealed land disposal to be a satisfactory and practical method of utilizing liquid digested sludge and recommends the following application rates. They indicate that it is possible to obtain results similar to that received from commercial fertilizer with application rates as low as 25 tons of dry solids per acre each year, while loadings up to 100 tons of dry solid per acre per year will not impair crop growth. These loading rates are equal to 100,000 and 400,000 gallons of 5 percent sludge per year. Much lower application rates may be used when the sludge must be purchased and hauled long distances and also local soil conditions may decrease the need. Another authority indicates that partly dried sludge could be applied at rates of 10 - 40 tons per acre. This is equivalent to 10,000 to 40,000 gallons of liquid sludge per acre per year.

When considering a particular field, the Department of Agriculture or O.A.C. should be contacted in relation to evaluating the fertilizer need if maximum results are desired. Potash and/or lime or other constituents may have to be added to supplement the sludge if fertilizer requirements are to be fully met.

SLUDGE QUANTITIES

The volume of sludge resulting from a sewage treatment plant will depend on a number of factors. Below are listed three of these factors:

1. Raw sewage strength and quantity

2. Degree of treatment afforded in the plant

3. The type of treatment given the resultant raw sludge

The sewage strength is measured in terms of B.O.D. and suspended solids. The strength can vary greatly depending on the type of industry served, sewer infiltration, and whether the individual water services are metered. The last two items will have an inverse effect on the volume of wastes received. Where only dry industries inhabit a municipality, an estimate may be made of sewage quantity by allowing 0.17# of B.O.D. and 0.20# of s.s. per capita per day.

The degree of treatment afforded by a sewage treatment plant will effect the quantities of sludge produced. The following table is presented as a general guide on plant efficiencies.

Percent Removal Table

	5 Day B.O.D.	Suspended Solids
Primary settling	30%	50%
Modified aeration	70%	70%
Single stage high rate trickling filter	85%	85%
Standard activated sludge S.T.P.	90%	90%

The processing of sludge in a digester will reduce the sludge quantity. Approximately 50% of the volatile solids can be removed in the digestion process.

Sludge volumes and thus moisture content are effected by the type of treatment process utilized in a plant. Primary digested sludge is coarse and can be concentrated to a dry solids content of 5 - 8 percent in the liquid form and 30 - 40 percent in a filter cake. Whereas digested activated sludge, being much finer, will have a dry solids content of 3 - 5 percent and 20 - 25 percent respectively.

The quantity of sludge generated in an activated sludge treatment plant will, in part, depend on the relationship between the sludge created by the ingestion of organics in the sewage, and the degree of endogenous respiration.

Sample calculations are submitted as a guide in determining sludge quantities.

Type of waste - domestic sanitary sewage

Influent 5 day B.O.D. - 200 p.p.m.

Flow to plant - 2.2 M.G.D.

Primary Treatment

Assume 50% s.s. removal

$$\frac{50\%}{100} \times 200 \text{ p.p.m.} \times 2.2 \text{ M.G.D.} \times 10\# = 2200\# \text{ of dry solids}$$

Liquid sludge, undigested, 6% solids,

$$2200\# \times \frac{100}{6\%} \times \frac{1}{10\#} = 3660 \text{ G.P.D.}$$

Liquid sludge digested, 6% solids, 40% reduction

$$2200\# \times \frac{100}{6\%} \times \frac{1}{10\#} \times \frac{60\%}{100} = 2200$$

Raw sludge cake, 70% moisture

$$2200 \times \frac{100}{30\%} = 7300\# \text{ of cake}$$

Digested sludge cake, 40% reduction 70% moisture

$$2200 \times \frac{100}{30\%} \times \frac{60}{100} = 4400\# \text{ of cake}$$

Activated Sludge Complete Treatment Plant

Assume 90% s.s. removal and no gain or loss due to biological activity.

$$\frac{90\%}{100} \times 200 \text{ p.p.m.} \times 2.2 \text{ M.G.D.} \times 10\# = 3950\# \text{ of dry solids}$$

Liquid sludge, undigested, 4% solids

$$3950 \times \frac{100}{4\%} \times \frac{1}{10\#} = 9900 \text{ G.P.D.}$$

Liquid sludge, digested, 5% solids, 40% reduction

$$3950 \times \frac{100}{5\%} \times \frac{1}{10\#} \times \frac{60\%}{100} = 4700 \text{ G.P.D.}$$

Raw sludge cake, 75% moisture

$$3950 \times \frac{100}{25\%} = 15{,}800\# \text{ of cake}$$

Digested sludge cake, 40% reduction, 75% moisture

$$3950 \times \frac{100}{25\%} \times \frac{60\%}{100} = 9500\# \text{ of cake}$$

VEHICLE DESIGN

In purchasing a vehicle the quantity of sludge to be removed must be known as well as the length of haul, the time operator is available, and the other uses the truck will be put to. For small plants, finances will greatly limit the type of vehicle which may be obtained. Cost may vary from $4,500 for a 10,000 G.V.W. truck to $15,000 for a 42,000# unit. The first unit might be equipped with a 600 gallon tank while the larger truck may have a 2400 gallon tank.

Department of Transport regulations limit the loading on a two axle unit to 28,000 pounds G.V.W. and on a three axle tandem rear axle vehicle to 42,000 pounds.

Vehicle Loading Table

	G.V.W.	V.W.	Pay Load
Two axle	28,000#	13,000#	1500 IG or 7½ cu. yds.
Three axle, tandem	42,000#	18,000#	2400 IG or 12 cu. yds.

For operation under varying conditions the three axle vehicle should have a tandem drive unit with a differential lock out. The two axle vehicle is most satisfactory when equipped with four wheel drive and differential lock out.

Liquid Sludge

Sludge tanks should be constructed with an oval or round cross-section. Structural cracks will develop in the thin plate if a square design is followed. The use of 12 guage high tensile steel plate will provide corrosion resistance and more than adequate strength.

The use of a top operated valve, with valve located in the tank itself, for the sludge discharge line, will prevent freeze up during winter operation. When a four inch gate valve is used the pipe between the tank and valve and the valve itself must be insulated and also provided with a heating element. The tank must also be constructed with adequate internal baffles, inspection and loading manholes, and vacuum relief valves on the top.

Example of Selecting Tank Truck

As per previous example the truck is to serve a 2.2 M.G.D. sewage flow to an activated sludge sewage treatment plant. Sludge digestion is provided.

Sludge load - 4700 I.G.D.

4 day week $\frac{4700 \times 7}{4}$ = 8200 I.G.D.

Short haul, 10 loads

$\frac{8200}{10}$ = 820 gallon tank

Long haul, 6 loads

$\frac{8200}{6}$ = 1360 gallon tank

The calculations indicate need for a 800 to 1400 gallon tank, depending on length of haul, on a two axle vehicle, a four wheel drive unit might be advised.

Sludge Cake

Sludge cake should, if possible, be handled in a dump box equipped with a water tight gasket on the rear gate. The sides should have more than normal free board allowing one cubic yard of volume for each ton of cake to be carried. A heavy duty hoist is advised to enable spreading with box up while travelling over rough terrain.

PUBLIC RELATIONS

When disposing of sludge great care must be taken to ensure that nuisances are not created. Outside the restrictions set out under the section "Public Health Considerations", further common sense rules must be followed to prevent obnoxious odour complaints. When a municipality receives numerous complaints regarding refuse and sludge disposal, very restrictive regulations may be inacted. One town in Ontario has two townships surrounding it, both of which will hardly let a sludge truck leave the town itself. The restrictive regulations in these townships no doubt were passed because of numerous well founded complaints.

Raw Sludge

When disposing of raw sludge great care must be taken to ensure that only very isolated dumping areas are used and then the applied material must be ploughed in very quickly. Winter weather will prevent odours, but it may be difficult to work the

material into the soil early in the spring and so, obnoxious odour might well develop at that time. In general, raw sludge products must be very carefully handled. It might be advisable to dispose of raw sludge at a land fill project; there daily coverage can be provided.

Digested sludge is less odourous and therefore immediate coverage is not as important. Liquid sludge can be spread evenly and thinly; where quick drying is possible, on sandy and elevated dry land, it may be disposed of close to homes. Nevertheless, great care must be taken that obnoxious odours do not carry to residences and furthermore only well digested liquid sludge can be considered in this category.

Digested sludge cake must be handled more carefully. It is difficult to spread thin and therefore wet lumps of material may emit some offensive odours. If possible this material should be worked into the soil soon after spreading.

General

Once careless practices have aroused the publics' ire, it is very difficult to restore their confidence. When a municipality is handling their own sludge haulage the driver should be instructed to exercise utmost care in selecting disposal sites. If complaints are received they should be investigated immediately and the reported problems resolved with dispatch. The vehicles must be kept in a clean and well painted condition. Valves must not leak and where sludge cake is being handled the rear gate of the dump box must be water-tight. High dump box sides will prevent the cake spilling to the roads and also hide the load from view. Supervisory personnel should make periodic checks on the operation by having the driver record his disposal sites in a daily log and actually visiting these sites from time to time. One good reason for handling your own disposal operation is that better control can be maintained in relation to protecting your reputation.

Where the sludge is removed under a contract, the contract document should outline disposal practices so as to protect your future operations.

Again we note, once a good reputation is lost it is very difficult to recover.

DISPOSAL PROGRAMME

Many aspects regarding sludge disposal have been reviewed. A few points not previously covered and others requiring further mention are considered here

When setting up a sludge disposal programme a decision must be made whether or not to let a contract for sludge haulage.

The contract method is often more economical but as previously mentioned it is difficult to maintain control over public relations. Further, when sludge is to be removed from a vacuum filter operation or a digester system containing fixed covers and gas collection equipment steady day to day removal must be assured. Often contractors will not supply a constant service.

In operating a sludge vehicle the municipality is well advised to obtain a competent driver. The driver should, under some supervision, be capable of planning a programme considering the various aspects mentioned in this lecture including public health considerations, odour control, public relations, and vehicle maintenance. A good driver will cultivate a demand for the sludge and he should ensure that dry accessible disposal sites are available for wet and between season weather. In promoting the product the term soil conditions should be used and the humus value of the sludge stressed. The Department of Agriculture will assist farmers in evaluating the needs of their land. Lime and possibly potash may be required to balance out the nutrient and pH needs of a soil where sludge is used.

The spreading of liquid sludge is not too difficult but the filter cake product will tend to form lumps. If the sludge lumps are too large, farm machinery may not be able to cultivate. Possibly, during the spring break-up sludge cake should be disposed of at land fill projects.

In closing, it is again stressed that raw sludge can be very obnoxious from an odour standpoint and therefore special precautions must be taken in its handling.

Tandem drive, 40,000 #G.V.W., 2000 I.G. Capacity Tank Truck spreading digested sludge in the winter time.

DEWATERING OF SLUDGE BY VACUUM FILTRATION

by

D. McTAVISH

Project Engineer, Plant Operations

An Address To
The Ontario Water Resources Commission
Intermediate Sewage Works Operators' Course
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GENERAL

Sewage treatment is concerned with the extraction of solids from sewage and the ultimate disposal of these solids. The dewatering of solids consists of plain sedimentation, sometimes digestion of the collected sludge in order to stabilize and render it inoffensive, drainage on sand beds or mechanical filtration and finally disposal on the land where evaporation continues the drying process.

Plain sedimentation removes 99.7 percent of the total water, digestion and sand bed dewatering or mechanical filtration remove approximately another 0.3 percent of the total water. Actually, about 0.04 percent of the total water present in the incoming sewage still remains after sand bed dewatering or mechanical filtration and this is sometimes removed by incineration.

Vacuum filters were utilized to dewater waste activated sludge at Milwaukee in 1925 and this was the first application of the filters in the sewage field in North America. However, "Drum Filters" were first invented in England in 1872 for cement slurries. The filters at Milwaukee used sulphuric acid as a coagulant, then chlorinated copperas, and finally ferric chloride.

A vacuum filter is a rotary drum under vacuum, suspended over a trough or vat which contains wet sludge ready for filtration. Appurtenances included with the filter are a vacuum pump, filtrate receiver, filter media, washing showers to clean the filter media, belt conveyors to remove the dried sludge and a hopper or storage area. Storage and pumping facilities for the coagulation chemicals and a mixing tank for the wet sludge and these chemicals are also required.

Various types of filter media have been employed since 1925. The ideal media is one that offers no resistance to the flow of liquid, but serves as a support for the solid particles forming the cake. The following materials are those most commonly used as filter media:

1. Fabric Covered Filters

The filter cloth is made from cotton, wool, felt, dacron, saran, polyethylene and many others. The synthetic fibres have longer life, greater yield in many cases and are easier to clean. Woven wool is an excellent media while new, but clogs with fine particles and conditioning chemicals very quickly. The expected life for wool is from 250 to 800 hours with an anticipated life of 10,000 hours for dacron and saran.

2. String Filters

These filters use a fabric media but the sludge cake is removed from the drum by strings which pass around it. No air blower or scraper is required.

3. Travelling Belt Filters

These consist of a stainless steel woven wire belt which serves as the filter media.

4. Coil-spring Filters

Two layers of coiled steel spring are placed in corduroy fashion around the drum. As the layers leave the drum, they are separated from each other and the filter cake is lifted off and discharged.

Raw, digested primary and/or activated sludge can be dewatered by vacuum filtration. The amount of chemical coagulants and the yield (pounds of dry solids per square foot of filter area per hour) depends to a large extent on the type of sludge to be filtered. For instance, raw sludge filters with less chemical coagulants than does digested activated sludge and usually at a higher rate.

SLUDGE CHARACTERISTICS AFFECTING FILTERABILITY

1. Size, Shape, Density and Charge of Solid Particles

Irregular shaped small particles tend to form a very tight mat with only a few voids for the migration of liquid. On the other hand, regular shaped larger particles tend to give a high ratio of voids. As a result, sludge consisting largely of small particles requires the greater amount of coagulant per unit weight of solid. It was found at the Hyperion plant in Los Angles that thermophilic digestion produced larger particle sizes than did mesophilic. Digestion at the higher temperature increased the filter yield three fold and decreased the chemical coagulant demand by 50 percent.

Sludge particles have been found to be negatively charged with a charge of about the same magnitude as that associated with proteins. In order that these particles will coagulate, it is necessary to neutralize this negative charge. Ferric chloride with three positive charges is most effective in neutralizing the negative charges of the sludge particles.

2. Compressibility of Solid Particles

Compressible sludges tend to deform with pressure and form a tighter filter cake. Thus an increase in pressure or vacuum will not result in a proportional increase in filter rate for a compresssible sludge. A compressibility of 1.0 means that the filter rate remains constant with an increase in pressure. In general, the compressibility of most sludges has been found to be approximately 0.8. Thus, a large increase in pressure will result in only a small increase in yield. This is demonstrated in Figure 1.

3. Viscosity of Filtrate and Sludge

The viscosity of the filtrate varies with temperature. At 55 degrees centigrade, for instance, the viscosity of water is one half of that at 20 degrees centigrade. However, it is probable that it would not be practical to attempt to raise the temperature of the sludge being filtered due to the cost involved and furthermore an elevation of temperature may produce changes in the sludge.

4. Chemical Composition

The chemical composition of sludge determines to a large extent the amounts and type of coagulating chemicals required. The greater the alkalinity of the sludge, the greater is the chemical demand; and the greater the amount of volatile matter, the greater is the chemical demand. These relationships are expressed as liquid demand (alkalinity demand) and the remaining demand is termed the solids demand.

Digestion of sludge increases the liquid demand, but decreases the solids demand. In decomposing the volatile portion of the raw sludge, the bacteria convert the putrescible compounds on the one hand to methane and carbon dioxide and on the other hand to ammonia. The carbon dioxide and ammonia combine in the sludge liquor to increase the alkalinity related to the remaining solids. In terms of solids present, the alkalinity is increased by 10 to 20 times what it was in the fresh sludge.

5. Solids Concentration

Increasing the solids concentration will increase the yield of a filter. The more concentrated the sludge is the less filtrate volume has to be removed per pound of filter cake deposited. Concentrating also lessens the liquid demand and, hence, lowers the amount of chemical coagulants required.

CHEMICAL CONDITIONING

The amount and type of chemical coagulants is determined by the physical and chemical characteristics of the sludge. Sludge solids are primarily thought of as complex colloid systems with water as the dispersion medium. The majority of these colloids remain in dispersion due to (1) electrical charges surrounding the particle and (2) the shell of water around the particle. In order to reduce the stability of these colloids one or both factors must be removed at least partially.

The stability of the electrical charge is due to (1) ionization and/or (2) adsorption of solution ions on the particle surface. Both of these factors are affected by pH, ionic strength and the type of ions in the surrounding water.

The stability of the shell of water around the particle is dependent on the structure of the molecule and the functional groups of the molecule.

Thus, any force which will neutralize the negative surface charges and/or bring about reactions with or among the functional groups can be expected to destroy the stability of the colloid systems in a sewage sludge.

The three positive charges of the ferric ion are most effective in the charge neutralization effect whereas some functional group reactions take place with $FeCl_3$ at an acid pH, others take place with lime at an alkaline pH, while a third can react with either chemical. Thus, coagulating chemicals are involved in a number of reactions with sewage sludge to make them filterable. The predominant reaction or set of reactions will vary with the type and source of sludge.

In the treatment of domestic sludges, there are three basic types of sludges. The first type, from plain sedimentation, is the easiest to handle since it is possible to obtain high concentrations both with the raw and the digested sludge. Sludges from trickling filters are more difficult to concentrate. Sludges from activated sludge treatment are usually the most difficult to concentrate due to their voluminous and spongy nature.

The bargraphs in Figure 2 show the relationship between ferric chloride demand and type of sludge. This graph was presented by Genter (1) and represents the amount of ferric chloride in pounds per 1000 persons required for the three basic types of sludges. The bar graphs are based on sludges collected from separate sewage systems handling domestic sewage at a flow of 80 gallons per capita daily.

The cross-hatched portion of the bar represents the solids demand while the dashed portion indicates the liquid demand. When the percentage of alkalinity on solids is relatively low, as with most of the fresh and elutriated digested sludges, most of the chemical is for solids requirement. The digested sludges display a reduction in solids demand due to the decrease in volatile to ash ratios but have a greater liquid demand due to the increased alkalinity resulting from the end products of digestion.

The graph for the activated sludges indicate that there is only a slight reduction in chemical requirements when these sludges are digested. Sludges of this type can decrease their chemical requirements appreciably by elutriation.

ELUTRIATION

Elutriation is a solids washing process. An elutriated sludge is one that has had the alkalinity of its fouled water (from decomposition products of digestion) reduced by dilution, sedimentation and decantation in water of lower alkalinity.

Elutriation involves four essential steps to improve the biochemical quality of sludge water:

1) Dilution with water of lower alkalinity than the fouled sludge water.

2) Mixing the sludge and dilution water to produce a more dilute solution of the dissolved decomposition products present in the fouled sludge water.

3) Sedimentation of the sludge solids in the more dilute solution.

4) Decantation of as much of the weaker, relatively clear solution as possible.

Since these steps usually result in incidentally washing the sludge solids free of entrained gases the elutriated sludge gains in specific gravity and concentrates to a higher degree than the digested sludge to elutriation.

For single stage elutriation, the following formula may be employed to determine the resulting alkalinity:

$$E = \frac{D + 2W}{R + 1}$$

where D is the alkalinity in the digested or stale raw sludge water before elutriation, E is the alkalinity of the elutriated sludge, W is the alkalinity of the elutriating water and R is the ratio of volume of elutriating water used to the volume of moisture in the sludge. Example:

Assume a digested sludge having a 4 percent solids and an initial alkalinity (D) of 3,000 ppm. The wash water alkalinity (W) is 120 ppm and the metered ratio of wash water to sludge volume is 4:1.

Then $R = 4 \div 0.96 = 4.17$

$$E = \frac{3,000 + 2 \times 120}{4.17 + 1} = 678 \text{ ppm}$$

This represents about a 78 percent removal of the initial alkalinity.

OPERATING VARIABLES

A number of variables confront the operator of a vacuum filter. He can change drum speed, amount of vacuum, chemical dosages and chemical ratios (if adding both lime and ferric chloride.)

Practically all of the proper techniques involved with these variables have been developed through operating experiences at a number of plants. Certain procedures with respect to vacuum filtration often seem trivial but have been found by experience to be quite important.

1) Sequence of Addition of Chemicals

This variable would appear quite insignificant to the laymen but it is quite important with respect to the total amount of chemicals required for coagulation. Experiments based on the Buchner funnel test indicate that for dosages for ferric chloride below 5.5 percent, it is important to add the ferric chloride ahead of the lime. Savings in ferric chloride and lime requirements in the order of 50 percent can be experienced by following this procedure. This is demonstrated in Figure 3.

2) Vacuum

Increasing the vacuum will result in a greater removal of moisture but with respect to filter yield vacuums in excess of 10 to 15 inches of mercury are not required. This is demonstrated in Figure 1. The sludge with a compressibility of 0.96 shows a 10 percent increase in yield with a four fold increase in vacuum. The other sludge, which is less compressible (0.63), shows a 32 percent increase in yield with a four fold increase in vacuum.

3) Mixing

Pilot plant studies have demonstrated the importance of mixing speed. Experiments have been made with paddle, flat blade turbine, propeller and curved blade turbine type mixers. It was found with all of these various types of mixers that the best flocculation occurred during a specific range of mixing speeds. The maximum flocculation was found to occur with the propeller and curved blade turbine type mixers. The paddle type mixers had a very narrow range of speeds for best results and flocculation fell off quite sharply with speeds on either side of this range (see Figure 4).

Sludges encountered from plant to plant will have different requirements for mixing speeds. In fact, in some plants the characteristics of the sludge vary from day to day. For this reason, it is advisable to be able to change the mixing speed to adapt it to the particular type of sludge being filtered.

Similarly, it is important to experiment with the use of the agitator. Excessive use of this equipment may break up the floc with certain sludges. Many have found only one or two minutes of operation every one-half hour is sufficient.

4) Cake Thickness and Drying Time

Increasing the cake thickness also increases the moisture in the filter cake. An increase in the drying time (decrease in yield) results in a decrease in the moisture of the filter cake.

5) Quantity of Chemicals

Underdosing of a sludge with coagulants will result in an incompletely coagulated sludge. However, data gathered from several operating filter installations

indicate that filter yield can be depressed by over-dosage of chemicals. Since chemical cost is often the largest single item in the cost of filter operation, unnecessary overdosage especially with ferric chloride can be unduly expensive.

OPERATING RESULTS

Some experiments have been carried out at one of our activated sludge plants with pickle liquor and ferric sulphate as replacements for ferric chloride. This type of sludge, without elutriation, is one of the most difficult to coagulate and filter.

It was found with both the ferric sulphate and pickle liquor that it was difficult to maintain a good floc. The floc produced was quite unstable and, as a result, the filtrate contained quite a high amount of solids. In fact, it was calculated from laboratory determinations on the filtrate, filter cake and incoming sludge that as much as 50 percent of the solids were being returned in the filtrate.

These solids will settle readily in the primary clarifiers and, hence, cause no problems there. However, this build up of sludge will increase filtering time, amount of chemicals required and thus increase the cost of filtering.

The formulae listed below can be used to determine the amount of solids being returned in the filtrate. It should be noted that it is necessary to know the volume of sludge being filtered and the moisture content of the incoming sludge, filter cake and filtrate before the volume of filtrate and amount of solids can be determined.

$$Vf = \frac{(Cb - Cs)\ Vs}{(Cb - Cf)}$$

$$Bs = \frac{Cb\ \ Vs}{10} \quad \frac{(Cs - Cf)}{Cb - Cf}$$

$$Fs = \frac{Vf\ \ Cf}{10} = \frac{Vs\ \ Cf}{10}\ \frac{(Cb - Cs)}{(Cb - Cf)}$$

$$Vb = \frac{Bs\ \ 100}{Cb} = Vs\ \frac{(Cs - Cf)}{(Cb - Cf)}$$

where Vf = volume of filtrate (gals)

Vs = volume of sludge to be filtered (gals)

Cc = % solids in sludge on belt.

Cs = % solids in sludge to be filtered.

Cf = % solids in filtrate

Bs = lbs. of solids on belt

Fs = lbs. of solids in filtrate

Vb = volume of sludge on belt (gals)

The filtering rates for various sludges are indicated below:

Type of Sludge	Average Yield psf/hr.	Range
Primary	10	8.5 - 11.5
Primary digested	8.6	4.5 - 12.5
Primary-digested-elutriated	8.3	3.2 - 13.0
Primary and Activated	3.0	
Primary and Activated-digested	2.7	2.3 - 3.2
Primary and Activated-digested-elutriated	3.1	

The chemical dosages involved in the filtering of sludges vary. They depend primarily upon the alkalinity of the sludge and the organic to fixed solids ratio. Some of the expected dosages for various sludges are shown in the following table:

	Fresh Solids		Digested		Elutriated Digested	
Type of Sludge	$FeCl_3$	CaO	$FeCl_3$	CaO	$FeCl_3$	CaO
Primary	1-2	6-8	1.5-3.5	6-10	2-4	
Primary & trickling filter	2-3	6-8	1.5-3.5	6-10	2-4	
Primary & Activated	1.5-2.5	1.5-4	1.5-4	6-12	2-4	
Activated alone	4-6					

The costs involved in the operation of vacuum filters include labour, power, water, chemicals and maintenance and repair. The costs indicated below do not include repair since the particular plants involved are quite new and repairs have been quite small. Only one plant in each category has been used in determining operating costs. It should be noted that these units have been in operation for only a short time and, as a result, the operating costs may be higher or lower than those that will be experienced in the future

Type of Sludge	Cost Per Ton of Dry Solids
Primary-digested	$ 7.21
Primary & Activated	19.70
Primary & Activated-Digested	14.10

FIGURE 1

EFFECT OF VACUUM ON FILTRATION RATE

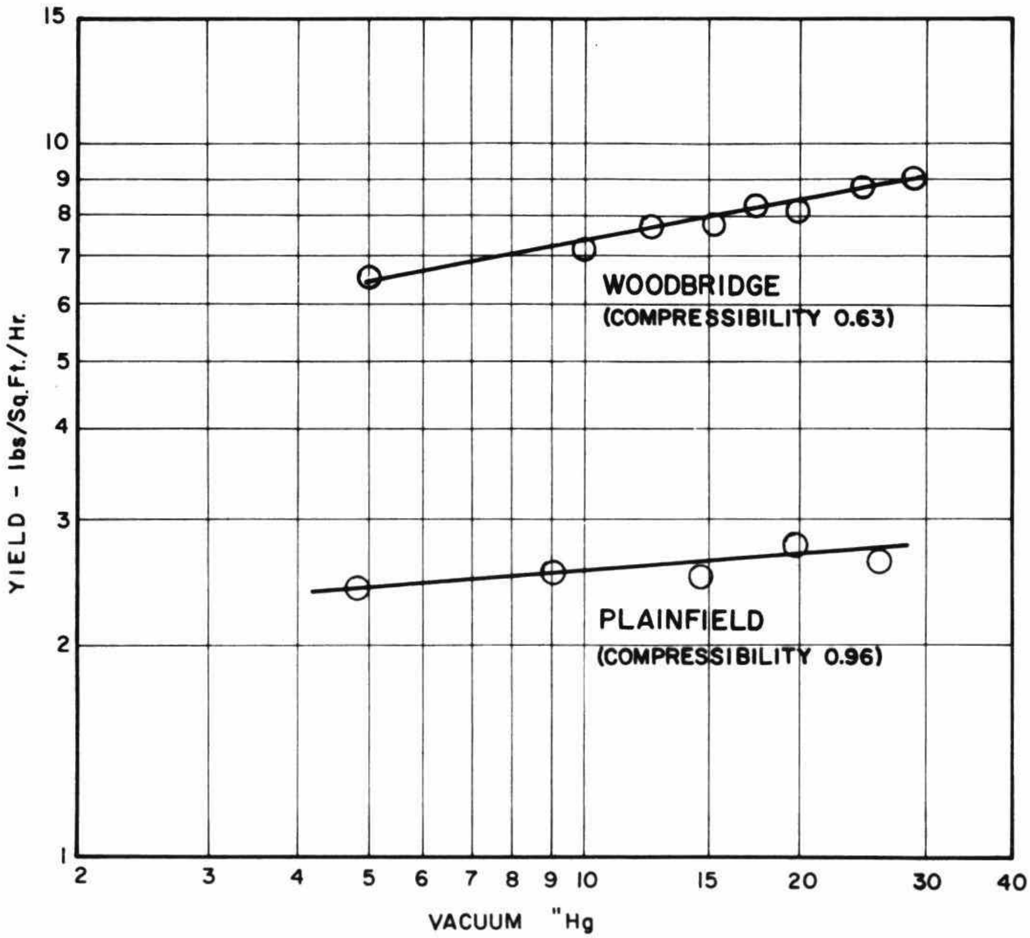
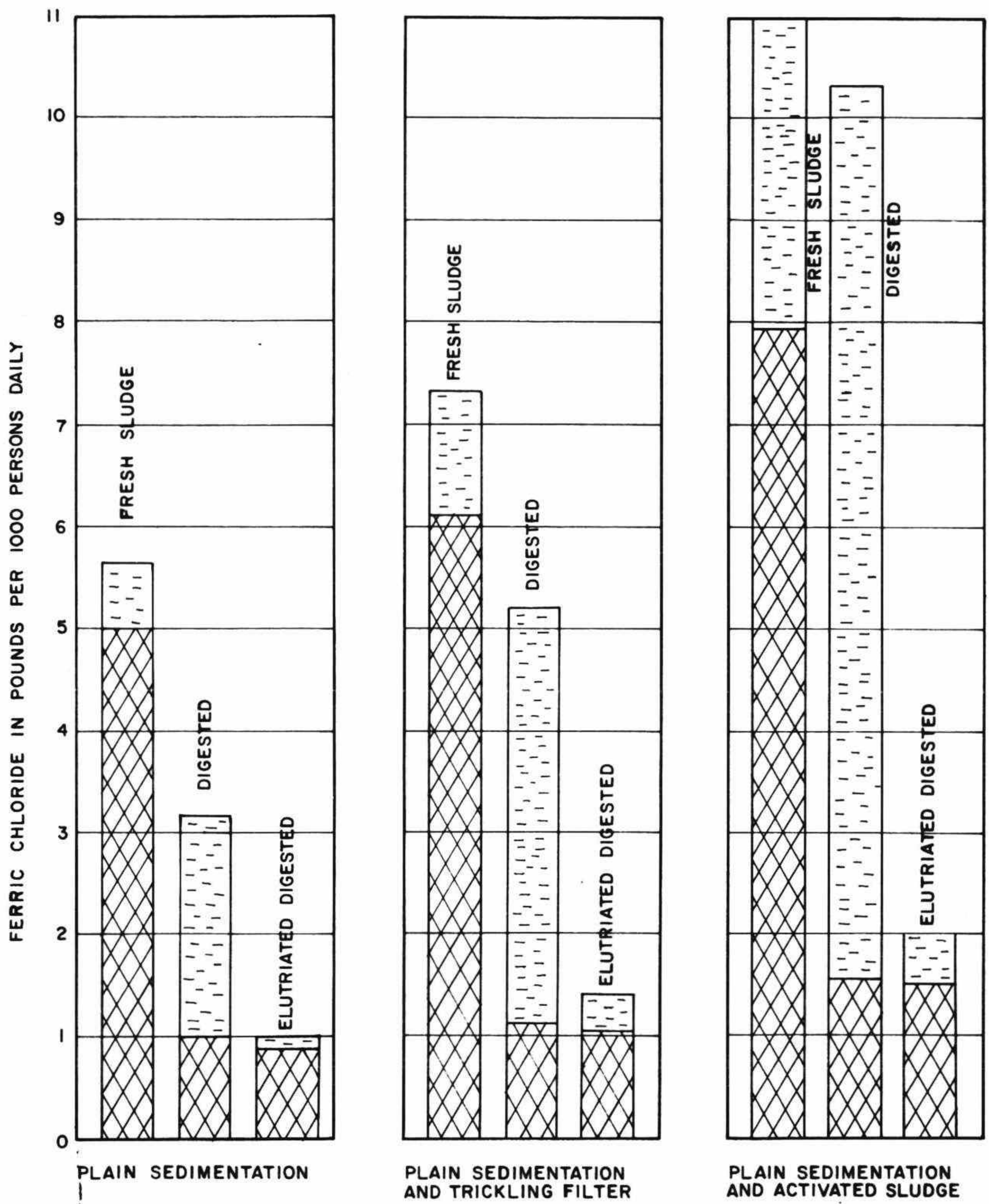


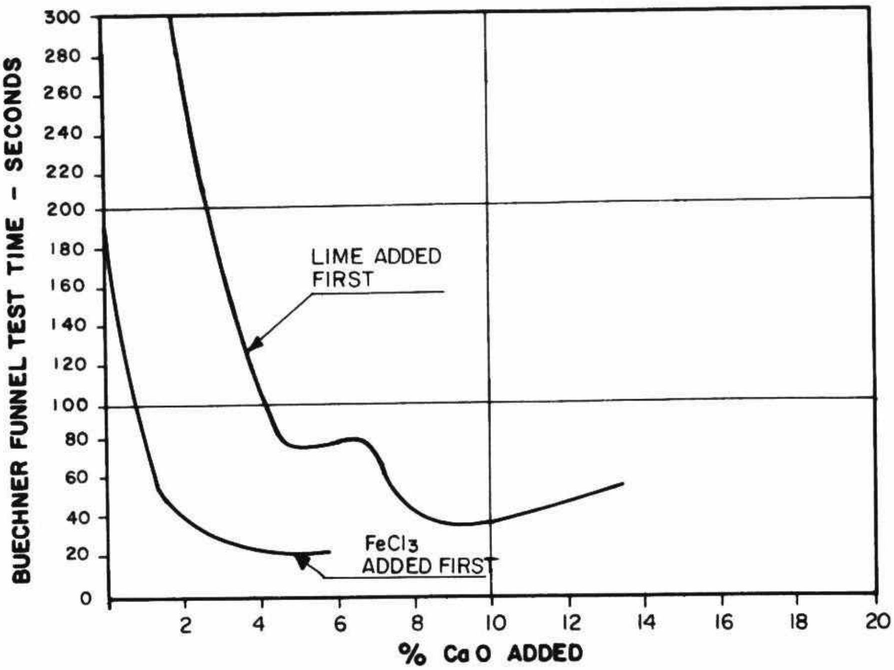
FIGURE 2



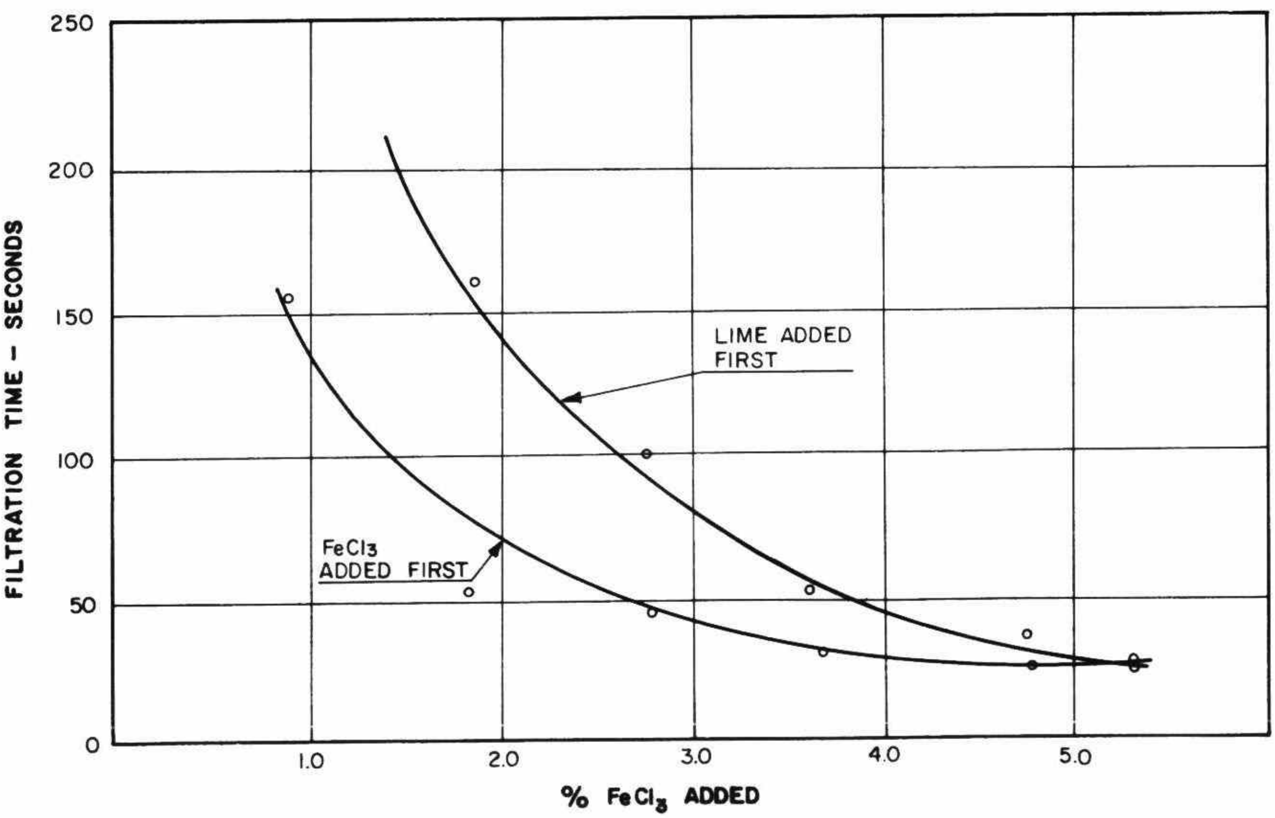
QUANTITY OF FERRIC CHLORIDE REQUIRED FOR VACUUM FILTRATION OF VARIOUS SLUDGES

FIGURE 3

EFFECT OF SEQUENCE OF CHEMICAL ADDITIONS



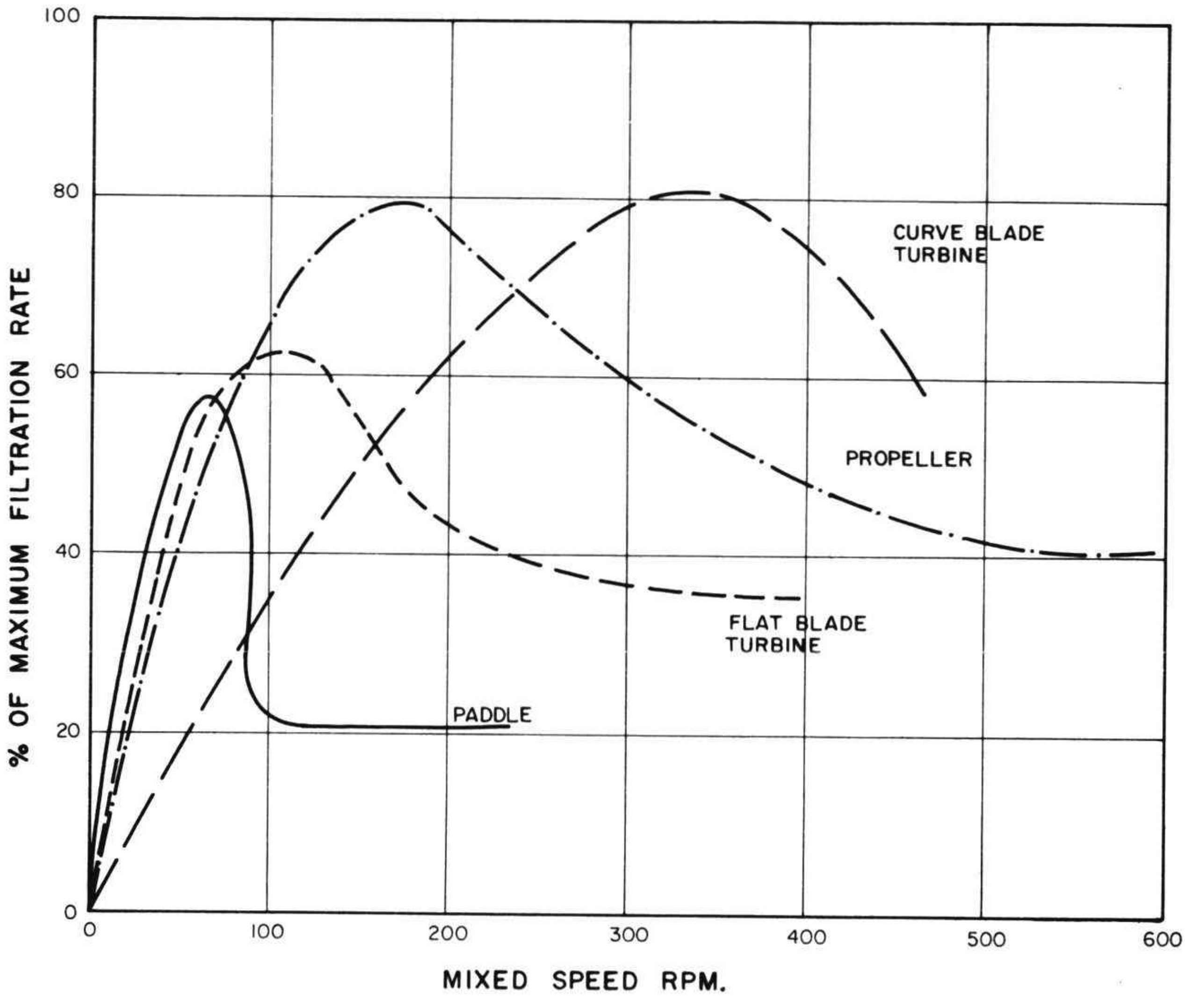
1) LIME REQUIREMENTS



2) $FeCl_3$ REQUIREMENTS

FIGURE 4

EFFECT OF MIXING SPEED ON FILTRATION



SUMMARY

Most of the progress with respect to vacuum filtration over the past thirty years has resulted from the operating experience of filter units at sewage plants. Some of the fundamental questions such as an understanding of the mechanisms of conditioning and the reactions involved have only been partially answered. Some work has been done in the theoretical field with respect to specific resistance as a means of expressing filtration results. Carman's theory in this field appears quite sound and has agreed with laboratory and full scale plant experiences. Laboratory techniques have been developed for determining the specific resistance of a sludge and the reader is directed to Sewage and Industrial Wastes, Vol. 28, No. 8. "Vacuum Sludge Filtration" by P. Coackley and B. R. S. Jones for information with respect to this approach.

The Buchner Funnel Test and the Filter Leaf Test are commonly used laboratory tests for determining chemical dosages. They serve as good indicators but unfortunately do not reflect small changes in filterability, and, as a result, the best chemical dosages must be determined at the filter unit. Although great advancement has been made in equipment design in the past decade in the dewatering of sludge, more work is needed to determine more effective coagulating agents which will permit a reduction in costs of filtering. Unless these methods can be found, it appears that other methods of sludge disposal may find their place in the sewage wastes field. The use of coagulating aids and some of the other polymers recently being marketed, as well as the possibility of using wastes from industry as coagulants, may be one method of reducing filtering costs.

The operator of vacuum filters can assist greatly in lowering costs by obtaining the maximum efficiency from his unit. He also should be willing to try new coagulating chemicals as it is difficult at present to correlate exactly between laboratory results and actual full-scale operations.

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DIGESTER OPERATION II

by

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An Address To
The Ontario Water Resources Commission
Intermediate Sewage Works Operators' Course
Toronto, Ontario
March 7, 1962

DIGESTER OPERATION II

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GENERAL

On the first course the lecture digestion of sludge covered the theory of digestion, design criteria, and operation. This lecture will concentrate on the operation aspect of the subject. The subject will be covered under three headings:

Digester Start up,

Single Stage Digestion,

and Two Stage Digestion.

DIGESTER START UP

The following basic procedures can be used in placing a digester in operation.

1. Ensure that all construction has been completed. When the digestion process has been started alterations and repairs on internal parts are difficult to make.

2. Fill all lines and tank with water; raw sewage may be used.

3. Add seed material if available. Supernatent or sludge from a properly operating digester is the only product which will be effective.

4. Heat tank contents to 90 to 95° F and maintain it.

5. Add raw sludge at a rate of .01 pound of solids per cu. ft. per day for an unseeded digester and a somewhat higher rate for a well seeded digester. The loading on a high rate process can be .25 pounds of solids per cu. ft. per day. The .01# of solids per cu. ft. per day is equal to 1000 gallons of 5% sludge to a 50,000 cu. ft. capacity digester.

6. Circulate the digester contents and maintain the temperature.

7. Check periodically for progress of digestion by determining the volatile acids and pH. As the process proceeds the quality of the gas can be checked. In a large plant the gas may be analized for $CO2$ content. Also the sludge alkalinity may be determined. Some lime may be added to control pH but if the volatile acids advance beyond 2000 ppm the sludge feed is reduced or stopped as required. Excess lime may inhibit the process.

8. Gradually increase raw solids loading on the basis of favourable trends. The loading must not be increased with volatile acids levels above 1000 ppm.

SINGLE STAGE DIGESTION

General

For simplicity, the subject single stage digester operation will be covered under four headings:

Loading,

Process,

Supernatent Selection,

and Digested Sludge Removal.

Loading

Ideal conditions would be met if the raw sludge could be pumped continuously to the digester. For various reasons continuous loading is not possible. Small plants receiving 8 hours per day of operator supervision may load the digester three times a day, say at 8 a.m., 12 noon, and 4 p.m. Where automatic pumping facilities are provided, the other extreme may be reached with loading being effected once each hour. Where supervision is provided on a 24 hour basis, manual control may dictate 6 - 8 pumping cycles per day. Excess water will be directed to the digester if too many pumping cycles are provided. When raw sludge must be pumped for some distance, to the digester, the sludge line must be filled with dilute

sludge before the pump is stopped. The next pumping cycle will direct the dilute sludge to the digester.

In a single stage operation the raw sludge is directed to the top half of the digester. As indicated in the flow diagram appended as Item 1, the raw sludge may be mixed with seed sludge leaving the heat exchanger.

Process

To maintain the process two main operating criteria must be met:

(1) Sufficient mixing must be afforded to bring the raw sludge in contact with seed material and also to maintain sufficient area free for the digestion process. Where mechanical or gas recirculation equipment is not available a careful check must be kept on the process to insure that a foaming condition is not created or that the reaction space left does not become too small. In a single stage unit, mixing facilities, if any, are designed to only mix the material in the top half of the tank. In practise such a design is near impossible. Thus it is difficult to obtain a concentrated sludge from a single stage digester operation.

(2) The second process criteria that must be considered is temperature. The ideal operating temperature for mesophilic digestion is between 90 - 95° F. A lower temperature may be used if excess digester capacity is available. Where mixing is not afforded by mechanical means or gas recirculation it is wise to maintain a considerable safety factor to allow for some blanket space losses.

To maintain a check on the process, various tests and records are required. The number of tests required or that can be economically performed at a plant will greatly depend on the equipment available and the size of the plant. Also where good mixing is afforded the chance of process failure is less; and therefore fewer tests would be required. A few of the tests, listed in order of importance, are as follows:

scum blanket depth,

digested sludge depth,

supernatent suspended solids,

volatile acids,

pH,

alkalinity,

gas compositions,

and sludge composition.

Records can be kept of:

sludge directed to digester,

sludge removed from digester,

temperature of sludge,

and mixer operating schedule.

Supernatent Selection

It is difficult to obtain a good supernatent from a single stage digester. Nevertheless an attempt should be made to remove at least some of the excess liquid from a single stage operation. Where mechanical mixing is practised the mixing devices are shut off for a period before the supernatent is withdrawn. Experience will show the quiescent period required to obtain a good supernatent.

When a variable level supernatent selection is provided, the supernatent is removed via the line proving to be most satisfactory. An example of a supernatent selector system is appended as Item 3. Withdrawal control is maintained in simpler installations by sleeve height adjustment. Other installations use valves to control the withdrawal process. In all installations a safety overflow is provided.

The suspended solids test is used to check on the efficiency of the withdrawal process. The actual test can be determined using a centrifuge for quick results, and the standard suspended solids test where complete laboratory equipment is available. The raw sludge directed to the digester may have a suspended solids concentration of 30 to 60 thousand p.p.m. Therefore, the supernatent suspended solids concentration should not be allowed to approach this figure or little headway will be made. A suspended solids concentrations of 1,000 to 3,000 might be considered permissible with the ideal level being below 500 ppm.

Digested Sludge Removal

The accumulated sludge should be removed as frequently as possible. As indicated previously, it is difficult to obtain a good supernatent from a single stage operation. Therefore, the resultant bottom sludge may not be too concentrated. In fact a 4 - 5% suspended solids content might be considered good for a sludge from a digester operation serving an activated sludge plant.

When the gas is utilized from a fixed cover operation, the digested sludge is best removed when the raw sludge is being pumped. This practise will maintain the gas pressure and prevent the intake of air which could create an explosive air-gas mixture. A large sludge withdrawal at one time could cause process failure due to lack of seed material.

Bottom withdrawal and depth samples, are tested to control the sludge withdrawal process. The suspended solids and volatile suspended solids tests are two criteria used to evaluate the operation.

TWO STAGE DIGESTION

General

The subject two stage digestion is covered under five headings:

Loading,

Process,

Sludge Transfer,

Supernatent Selection,

and Digested Sludge Removal.

Loading

When high rate complete mixing is practised, the raw sludge may be directed to any point in the first stage tank. Otherwise the loading procedure is similar to that used for the single stage operation.

A good two stage design will allow either or any tank to be used for the first stage or heated unit. An example of a two stage digester flow diagram is given as appended Item II.

Process

Where mixing devices are available they are operated to control scum blankets and inactive dead spaces. Most of the mixing is effected in the first stage tanks. Often mixing units are not installed in the second stage tank. The mixing devices may be operated either full or part time. When part time operation is desired the cycle is set up in relation to tests and observations of scum blanket formation and not on power saving. In some operations the mixers may only be used a few hours a day.

Where mixing cannot be effected on a routine basis, process failure may occur at any time. The active volume available for the digestion process can be greatly reduced by the formation of scum blankets and sludge banks. Foaming can occur when the scum blanket begins to digest. The scum blanket may be partly controlled by the use of compressed air to mix the tank contents. When using air for mixing, great care must be taken to insure that the explosive air - gas mixture is not ignited. Because of the hazard of this control measure, it can only be effected two or three times a year. If just once the designers had to remove the scum blanket from a digester using fire hoses, shovels, etc., they would think twice before designing a digester lacking positive mixing devices.

When heating units are available they are generally used to heat the contents of the first stage digestion tanks. Optimum mesophilic digestion is carried out at between 90 and 95° F. However lower temperatures may be used where excess digester capacity is available. The maintained temperature should be such as to provide some safety factor.

Sludge Transfer

Sludge can be transferred from the first stage digester by a number of means, three of which are as follows:

(1) Automatic transfer may be effected via an equalizing line, as shown on appended flow diagram Item II.

(2) Sludge may be transferred via the heating recirculating line.

(3) Bottom sludge may be pumped to the second stage unit.

The transfer programme should be set up to delay the removal of solids from the first stage unit. If possible top material is transferred when the mixing devices are off. Nevertheless frequent transfers must be made from the bottom of the first stage tanks. If this is not done, the bottom withdrawal line could become plugged with grit.

Supernatent Selection

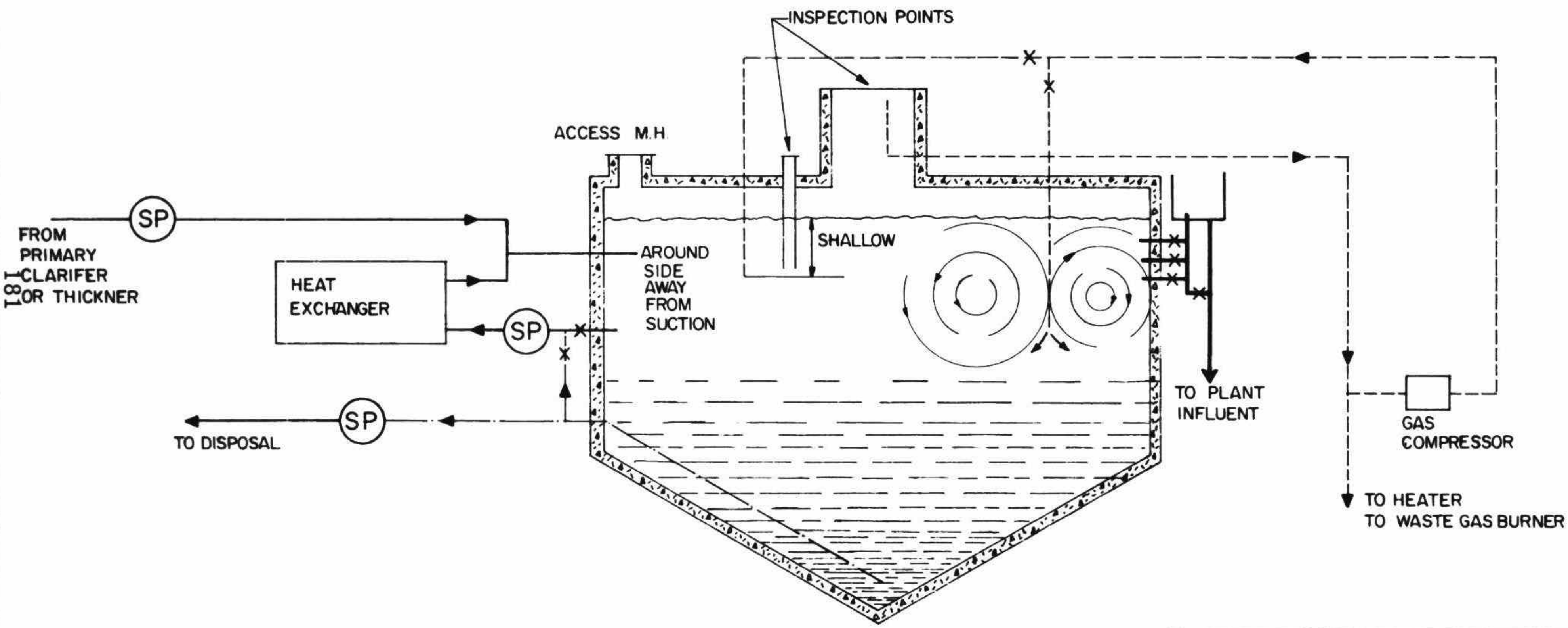
The supernatent is obtained from the second stage digestor. The supernatent can be selected automatically when a sludge transfer takes place or by design when the plant can best receive the extra B.O.D. loading. The type of selectors provided will of a necessity partly dictate the programme to be chosen.

Digested Sludge Removal

In a fixed cover installation the sludge must be removed in small batches. If this is not done, pressure will not be maintained.

When at least one floating cover is provided, the sludge settled in the second stage unit may be removed as convenience requires, large withdrawals will not cause process failure or a loss of gas pressure.

Sludge samples should be collected as indicated for the single stage operation. A two stage operation should provide a more concentrated sludge.



Legend

- SUPERNATENT
- GAS LINES
- SLUDGE WITHDRAWAL LINE
- SLUDGE LINES
- SP SLUDGE PUMP

ONTARIO WATER RESOURCES COMMISSION

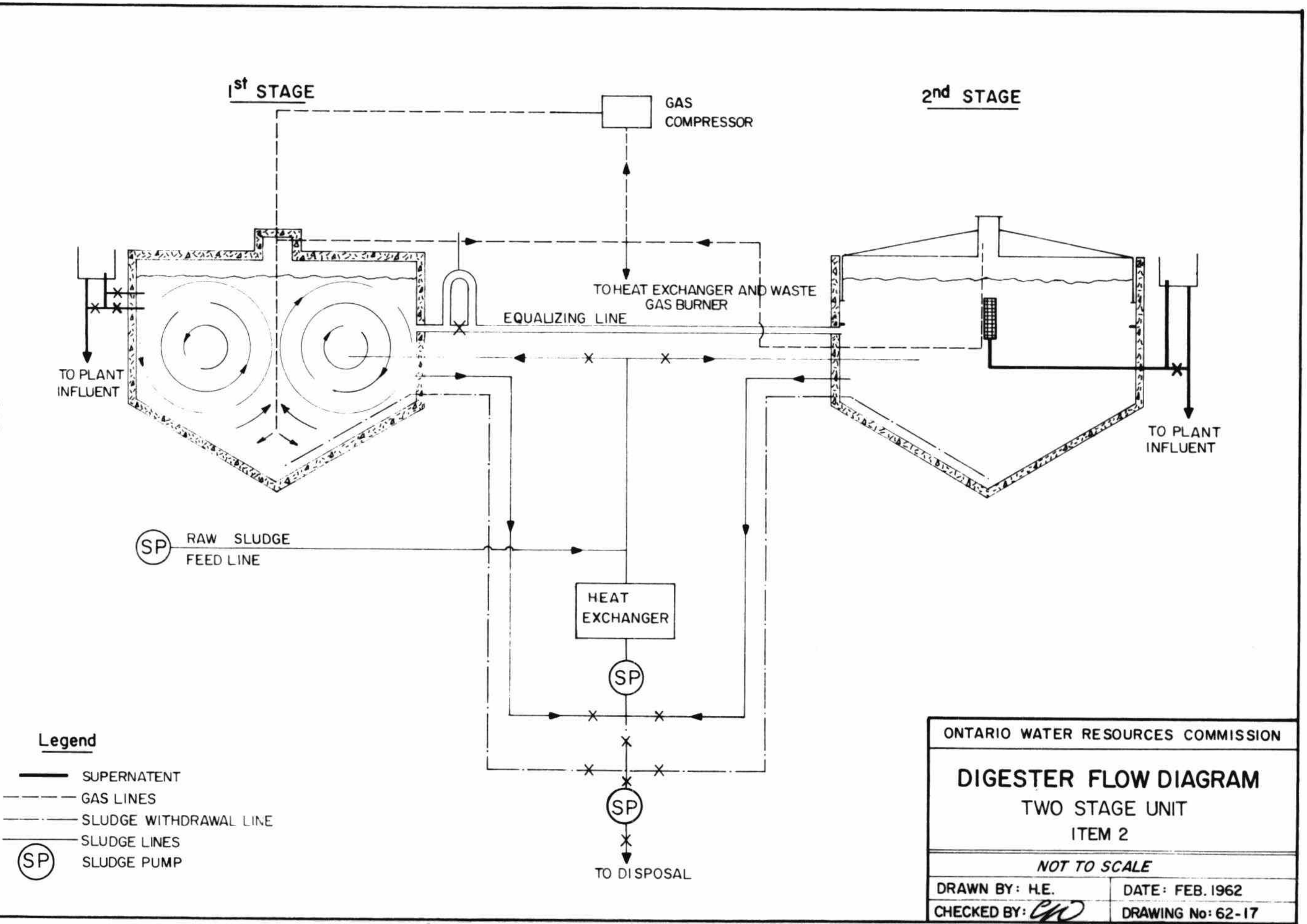
DIGESTER FLOW DIAGRAM

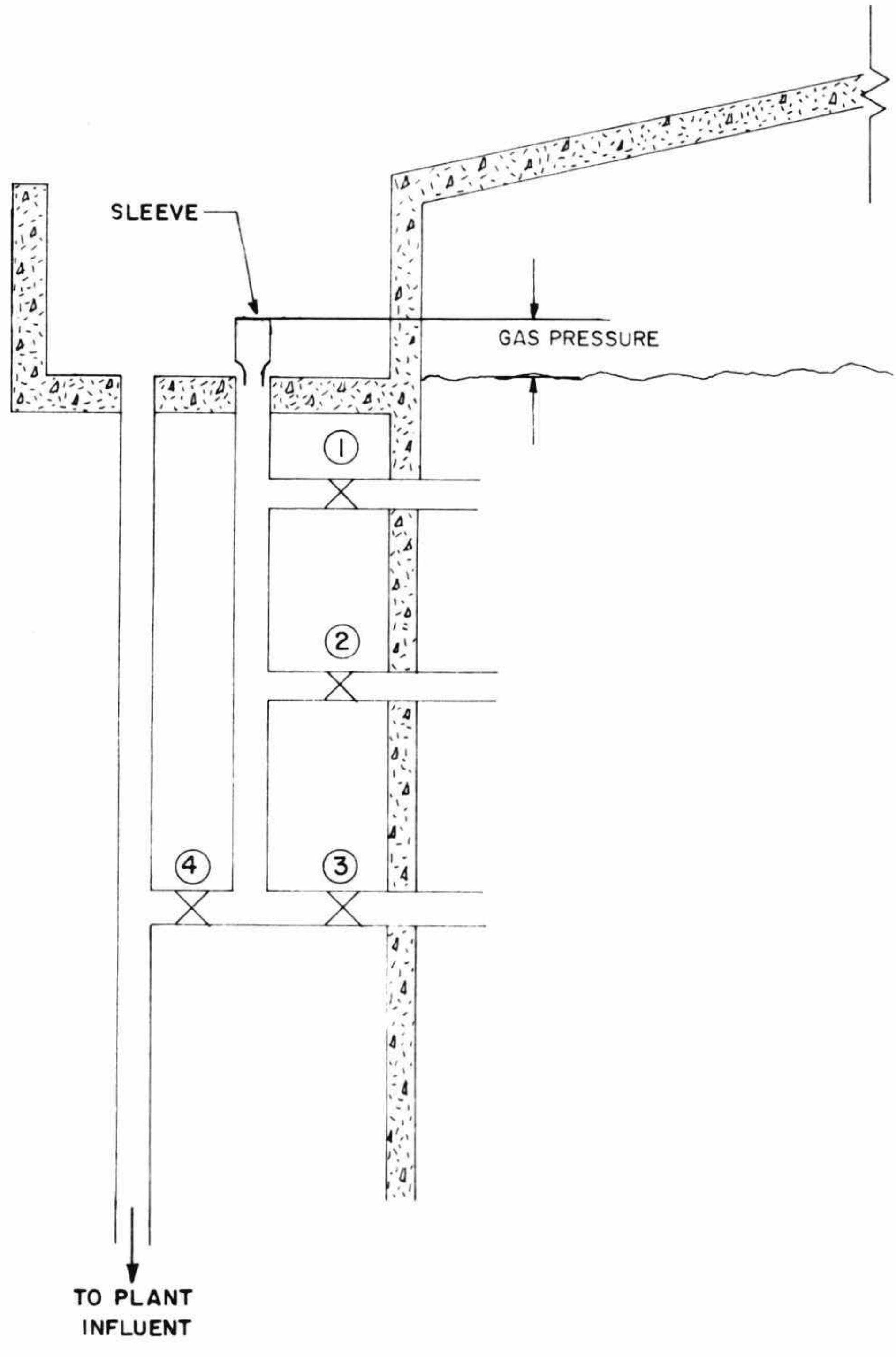
SINGLE STAGE UNIT

ITEM I

NOT TO SCALE

DRAWN BY: H.E.	DATE: FEB. 1962
CHECKED BY: CP	DRAWING No: 62-16


1st STAGE
GAS COMPRESSOR
2nd STAGE
TO HEAT EXCHANGER AND WASTE GAS BURNER
EQUALIZING LINE
TO PLANT INFLUENT
TO PLANT INFLUENT
SP
RAW SLUDGE
FEED LINE
HEAT EXCHANGER
SP
SP
TO DISPOSAL
Legend
SUPERNATENT
GAS LINES
SLUDGE WITHDRAWAL LINE
SLUDGE LINES
SP
SLUDGE PUMP
ONTARIO WATER RESOURCES COMMISSION
DIGESTER FLOW DIAGRAM
TWO STAGE UNIT
ITEM 2
NOT TO SCALE
DRAWN BY: H.E.
DATE: FEB. 1962
CHECKED BY:
DRAWING No: 62-17



ONTARIO WATER RESOURCES COMMISSION	
DIGESTER FLOW DIAGRAM SUPERNATENT SELECTOR ITEM 3	
NOT TO SCALE	
DRAWN BY: H.E.	DATE: FEB. 1962
CHECKED BY: G.V.	DRAWING No 62-18

LABORATORY DEMONSTRATION

THE CENTRIFUGE TEST & ITS APPLICATION

by

A. R. TOWNSHEND and R. J. NORTON

An Address To
The Ontario Water Resources Commission
Intermediate Sewage Works Operators' Course
Toronto, Ontario
March 8, 1962

LABORATORY DEMONSTRATION

THE CENTRIFUGE TEST AND ITS APPLICATION

by

A. R. TOWNSHEND and R. J. NORTON

INTRODUCTION

Considerable lecture time devoted to the activated sludge process has been spent in the presentation of loading factors and formulae which require determination of suspended solids in various sludges.

The centrifuge test has been referred to by Mr. L. South (Basic Course, pages 70 and 71) and by Mr. A. Townshend (Basic Course, page 78) as an acceptable, rapid method of estimating suspended solids at plants where filtering, heating and weighing equipment are not provided.

The centrifuge test is not a new concept in activated sludge treatment laboratory control. Mr. T. R. Haseltine suggested the use of an electrically operated centrifuge, provided with 15 ml. tubes, for quickly indicating the amount of suspended material in the mixed liquor as early as 1937. In recent years very little has been published to supplement the work done by Haseltine in 1937.

In Ontario, the Chicago Pump Company has drawn this test to the attention of plant operators by publishing a procedure in the operating manual supplied to all of its mechanical aeration plants.

PROCEDURE

The centrifuge comes with two 15 ml. graduated tubes. Both tubes should be filled to the mark with a well-mixed sample from the aeration tanks and inserted in the holders. <u>For uniform results the centrifuge must always be operated at the same speed for the same length of time</u>.

Speeds in the order of 1500 to 1800 r.p.m. are commonly used. For hand operated machines this corresponds to about two revolutions per second. Generally, the time of operation used is 3 or 5 minutes.

The centrifuge should be allowed to stop without breaking. The volume of solids contained in the bottom of each 15 ml. tube is then read. The average valve should be recorded as the result of the test.

Care should be taken to fill the 15 ml. tubes in the same manner for every test. It is considered better to dip the tubes in a well-mixed sample of mixed liquor rather than to pour the sample into the tubes.

Also, the sample of mixed liquor should be collected from the same point in the aeration tanks each day. It is usually taken from the effluent chamber of the aeration tanks as the mixed liquor enters the final settling tanks.

The test should be conducted at least daily. Since the suspended solids change during the day with sewage loading and hydraulic loading centrifuge tests should be made at the same time each day.

Laboratory glassware and sampling bottles should be cleaned thoroughly with soap and warm water after use.

APPLICATION

Before this test can be intelligently used for control purposes it is necessary to prepare a centrifuge vs. suspended solids conversion curve.

The operator must collect a number of duplicate mixed liquor samples at different times under different loading conditions and determine the suspended solids content by weight and volume (centrifuge test). If facilities are not available at the plant to determine the suspended solids by weight, the duplicate samples may be sent to the OWRC laboratory for analysis.

The two test results for each sample are then plotted on graph paper and a line drawn to fit the points. The graph can then be used for obtaining the weight of suspended solids from the centrifuge test.

Typical conversion curves are shown in Figure I. It may be seen that the slope and equation for these curves vary from plant to plant.

Relation to the Sludge Volume Index (SVI)

The Mohlman Sludge Volume Index is used to indicate the density and condition of the mixed liquor. It is determined by the following formula:

$$SVI = \frac{\%\ \text{settleable solids} \times 10{,}000}{\text{ppm. Suspended solids}}$$

Both the 30 minute settling test and the centrifuge test are volume measurements. It is reasonable to assume that a light sludge will give a high centrifuge reading as well as a high 30-minute settling test. This suggests that some of the variation found from plant to plant and from time to time in each individual plant may be overcome if the centrifuge test is also related to the Sludge Volume Index.

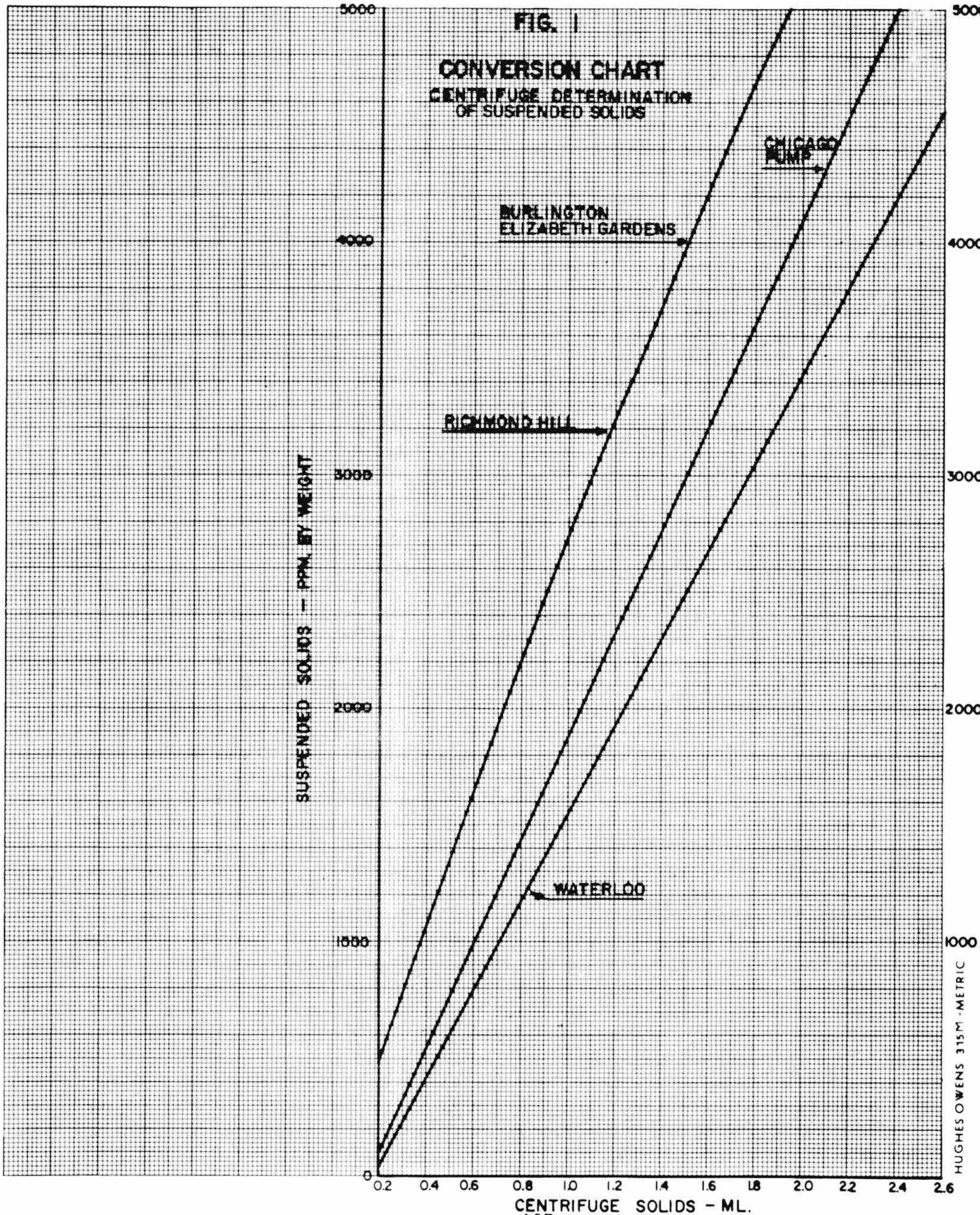


FIG. 1
CONVERSION CHART
CENTRIFUGE DETERMINATION OF SUSPENDED SOLIDS
CHICAGO PUMP
BURLINGTON
ELIZABETH GARDENS
RICHMOND HILL
WATERLOO
SUSPENDED SOLIDS – PPM. BY WEIGHT
CENTRIFUGE SOLIDS – ML.
0
1000
2000
3000
4000
5000
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
2.2
2.4
2.6
HUGHES OWENS 315M-METRIC

This concept developed in 1937 by T. R. Haseltine appears to have been lost in subsequent years. A typical diagram for determining suspended solids in the mixed liquor based on the centrifuge test and the Sludge Volume Index as reported by T. R. Haseltine is shown in Figure 2.

At first, it appears as if the operator must know the suspended solids to calculate the Sludge Volume Index to determine the suspended solids. However, the graph can be solved by trial and error since the 30-minute settling test can be performed by the operator as he does the centrifuge test.

Example I

Assume: (a) 30 minute settling test = 25%
(b) Centrifuge solids = 0.6 ml

From 45 and less SVI Curve (Fig.2)

ppm. = 3000

Therefore $\quad$ SVI. $= \dfrac{25 \times 10{,}000}{3000} = 83$

From 60 to 100 SVI Curve

ppm. = 2300

SVI. $\dfrac{25 \times 10{,}000}{2300} = 105$

From 170 to 210 SVI Curve

ppm. = 1950

SVI. $= \dfrac{25 \times 10{,}000}{1950} = 128$

It may therefore be concluded that the suspended solids concentration is about 2200 ppm.

Example II

Assume: (a) 30 minute settling test = 35%
(b) Centrifuge solids = 0.7 ml.

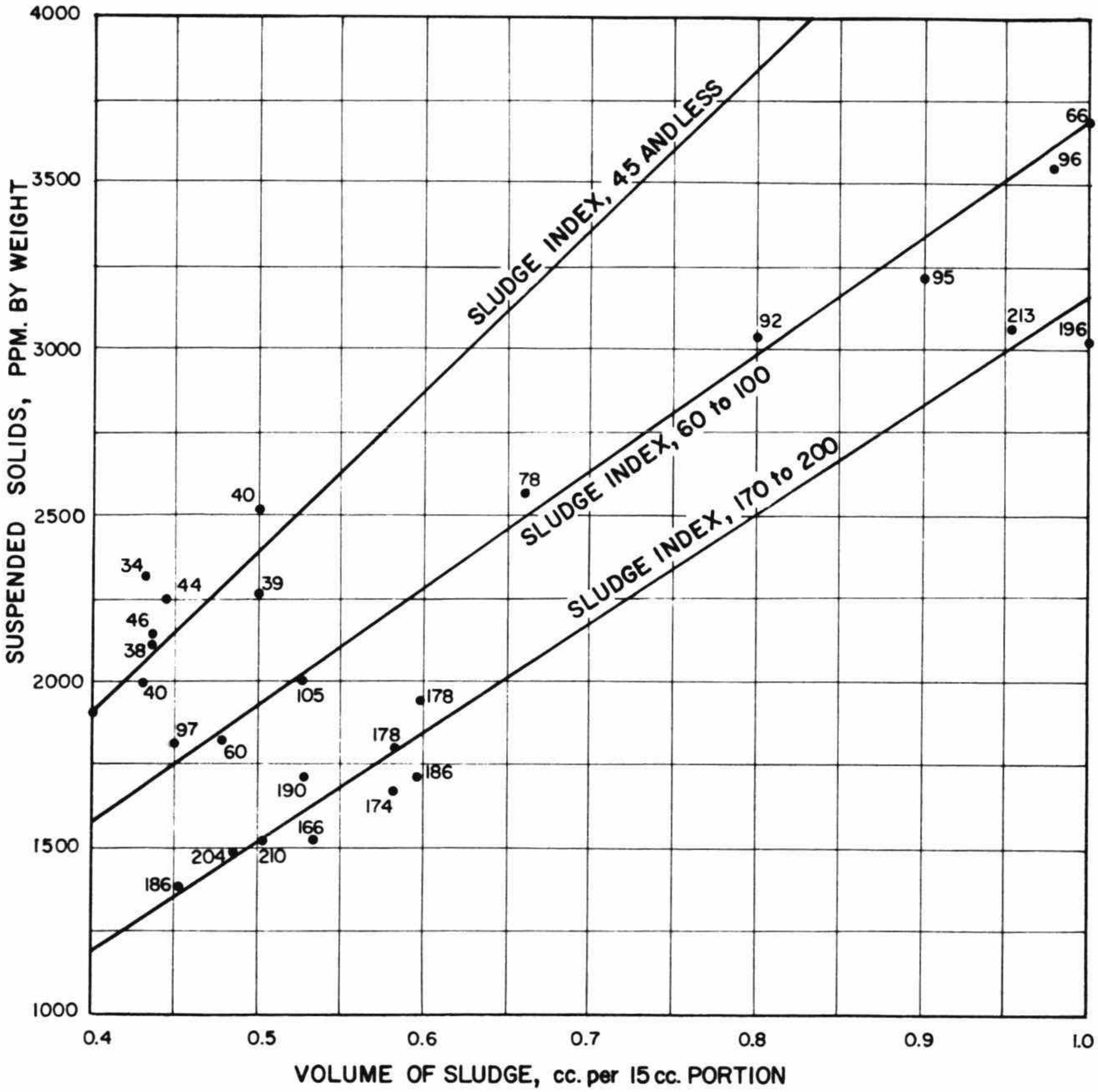
From 45 and less SVI Curve (Fig.2)

ppm. = 3550

SVI. $= \dfrac{35 \times 10{,}000}{3550} = 97$

FIGURE 2

TYPICAL DIAGRAM FOR DETERMINING SUSPENDED SOLIDS IN THE MIXED LIQUOR IN AERATION TANKS



From 60 to 100 SVI Curve

$$ppm. = 2650$$

$$SVI. = \frac{35 \times 10,000}{2650} = 132$$

From 170 to 210 SVI Curve

$$ppm. = 2175$$

$$SVI. = \frac{35 \times 10,000}{2175} = 160$$

It may therefore be concluded that the suspended solids concentration is about 2400 ppm.

By introducing the Sludge Volume Index the concentration of suspended solids only increased from 2200 ppm. to 2400 ppm. (200 ppm.) for a corresponding increase of 0.10 ml. by the centrifuge test.

From the graphs shown on Figure I an increase in centrifuge solids from 0.6 ml. to 0.7 ml. would indicate an increase in suspended solids at Fergus, Richmond Hill and Waterloo of 600, 220 and 150 ppm. respectively.

Until Sludge Volume Index Curves are established for plants in Ontario it is not possible to demonstrate properly the advantage of this refinement in technique.

SUMMARY

The use of an electrically operated centrifuge to indicate the amount of suspended material in activated sludge mixed liquor has been demonstrated.

Conversion Charts based on (a) Centrifuge tests and (b) Centrifuge tests and Sludge Volume Index have been presented.

An attempt has been made to show that the latter chart corrects for some of the variations that occur with this procedure.

CONCLUSIONS

With the data presently available it is not known whether all plants with the same Sludge Volume Index will give the same Centrifuge curve (with respect to the slope and coordinates) or whether each plant will have its own curve.

The operators of activated sludge plants in Ontario with centrifuge equipment can greatly increase the limited information presently available by developing SVI - centrifuge curves for their plants and submitting copies to the Ontario Water Resources Commission for study.

LABORATORY DEMONSTRATION

ANALYTICAL METHODS FOR DISSOLVED OXYGEN

by

C. E. SIMPSON

Supervisor, Chemical Laboratories

An Address To
The Ontario Water Resources Commission
Intermediate Sewage Works Operators' Course
Toronto, Ontario
March 8, 1962

LABORATORY DEMONSTRATION

Analytical Methods for Dissolved Oxygen

by

C. E. SIMPSON

Two methods are in general use, the Winkler and the Miller methods. For those intending to carry out either of these tests for dissolved oxygen we strongly suggest acquiring one of the texts previously recommended. These include "Standard Methods for the Examination of Water and Wastewater" Eleventh Edition, and "Analysis of Water and Sewage" by Theroux, Eldridge, and Mallmann, both of which contain detailed instructions for the Winkler test and its modifications.

A sheet of brief instructions for the Miller method was distributed at the demonstration. This is presently under review and the improved copies will be available on request during the final course in this series.

This written summary will therefore include only those points covered during the demonstration which are thought to supplement the above texts.

The Winkler method is the more accurate, with modifications developed to overcome many of the interferences commonly encountered in dissolved oxygen measurement in sewage. To yield the accuracy of which the method is capable, special precautions have to be taken in sampling to avoid exposure to air which can easily upset the original dissolved oxygen content in the sample. A sampling device which provides for flushing of the sample bottle and its sealing under water should be used. Plans for a sampling device of this type are included in both of the above texts. These provide a sample which has not contacted air, and which can be retained in this condition.

The basic chemical reactions involved in the Winkler test are as follows. The manganous compound added, reacts under alkaline conditions with the dissolved oxygen that is present to form a manganic compound. Thus, the dissolved oxygen is trapped as a manganic precipitate. Upon the addition of acid, this manganic precipitate reacts with the potassium iodide previously added, releasing an equivalent amount of free iodine. The manganic compound is converted back into its original manganous form. The released iodine thus traps the original dissolved oxygen in a stable measurable form. The amount of iodine is then measured by titration with thio-sulphate using starch to indicate the final stage of the end point.

These reactions are oxidizing and reducing reactions and if any other oxidizing or reducing materials are present they may react at any stage in any of the reactions to 'interfere', or cause low or high results. It is good practice to titrate the iodine immediately, since the longer this stands the more possibility there is of interference at this stage.

INTERFERENCES COMMON IN SEWAGE

Oxidizing substances interfere in the same manner as would extra oxygen. That is, they give high results. Interferences of this type include exposure to air at any stage in the reactions up to the formation of the iodine. Chlorine is an oxidizing compound and reacts to release additional iodine giving high results. A number of other oxidizing materials may occur in sewage and these may interfere with this test. An oxidizing substance which is commonly present in well treated effluemts and in streams is nitrite.

Reducing substances on the other hand tend to react as if there had been a loss of oxygen and thus give low results. The material present in sewage which gives the greatest problem in this respect is organic solids, especially the suspended organic solids. These may react either with dissolved oxygen in the presence of alkali or they may react with the released iodine at the acid stage. In either case, they give low results. Another possible reducing substance found in sewage is sulphides.

METHODS TO OVERCOME INTERFERENCES

Exposure to Air:

This is overcome by proper sampling techniques and by the deft addition of reagents below the surface followed by an immediate careful sealing of the sample bottle. Air bubbles must be excluded from the sample bottle until the iodine stage is reached.

Chlorine:

There is a modification of the Winkler test for use in the presence of chlorine but it is difficult and somewhat unreliable. The best solution is to avoid, wherever possible, taking samples which contain chlorine, that is, to sample either ahead of, or some distance below the chlorine contact chamber.

Nitrites:

The Alsterburg modification of the Winkler method, the one commonly used, is designed to overcome nitrite interference. The Azide used in this modification reacts with and breaks down nitrites at the acid stage. It also overcomes interference due to small quantities of ferrous iron.

Sulphides:

This is not too much of a problem since sulphides and dissolved oxygen are basically incompatible. That is, they react with one another until one of them is completely used up. Thus if there is a sulphide smell in the sample it is unlikely that dissolved oxygen is present. Rather than proceed with the prescribed modification which is the same one that is used for chlorine it might be best to check whether dissolved oxygen is present merely by the addition of methylene blue either to the sample bottle or to the plant at the point of sampling. If the methylene blue retains its blue colour it indicates that some dissolved oxygen is present. If the methylene blue turns colourless it confirms the absence of dissolved oxygen.

Organic Solids:

First of all, do not delay the titration since the free iodine can be used up by these organic solids. A number of methods or modifications have been worked out to try and overcome organic solids interference. The simplest of these is merely to proceed quickly with each stage of the test without waiting for the manganic precipitate to settle out. Where samples are very turbid this alone will not be sufficient. The best approach in these cases is to try and remove the suspended organic solids. This may be possible by settling alone but it is common to use a flocculating agent to weight the suspended particles and carry them more quickly to the bottom. The clear supernatant can then be siphoned off. Copper sulphate solutions are often used and have an added advantage in the following case.

Aeration Tank Liquors:

The problem here is that the biological activity in these tanks is so great that during the interval between the taking of the sample and the completion of the test reactions, a good portion of the oxygen present at the instant of sampling may have been used up. A copper sulphate solution added to the sampling bottle <u>before</u> the sample is taken will instantly mix with it. The copper will kill the bacteria and cause the biological activity to cease. Sulphamic acid may be added as well, since it is both a bactericide and also acts to destroy nitrites. The copper sulphate reacts with the alkalinity in the water and forms a copper hydroxide floc and this carries the particles down leaving a clear supernatant.

To Fix Samples:

The copper sulphate treatment above can be used, or for complete reliability when it is impossible to test the samples immediately a combination of sulphuric acid and sodium azide can be used. This completely arrests bacterial action and prevents nitrite interference. When sulphuric acid is used a compensating additional amount of alkaline azide reagent must be added to the sample when the test is performed.

Caution:

<u>The reagents used in this test are extremely corrosive</u>. Protective clothing and safety measuring devices should be used. For instance, if pipettes are employed, bulbs rather than the mouth should be used to draw the reagents up into the pipette. The reagents in the Miller method are not quite so corrosive. The worst is the alkaline tartrate which consists of 12% caustic (lye).

MILLER METHOD

The basic chemical reactions involved in the Miller test are as follows. The ferrous iron added (as ferrous ammonium sulphate) reacts directly with dissolved oxygen in the presence of alkali, using the dissolved oxygen up, and forming the ferric form of iron. This is insoluble in alkali and tends to form a brown precipitate which would obscure the end point. Tartrate is added and combines with this ferric product to keep it in solution as a colourless soluble complex. As long as dissolved oxygen remains the methylene blue stays coloured. At the point where all the dissolved oxygen is used up this indicator dye turns colourless.

The main use of this method has been in the field for stream surveys where the possibility of interference is less likely. Here the simplicity of this method and the absence of extremely corrosive agents out-weigh its inherently lower accuracy.

This is because this method depends to some degree on the technique of the person using it. Thus one operator might consistently arrive at higher or lower results than another even though both their results would be reproducible. For this reason, it is wise for each person performing the test to standardize his technique after practice and if necessary work out a correcting factor to apply to his results. Another problem is that the ferrous ammonium sulphate reagent may gradually lose its strength. It is not necessary to discard this reagent on this account, since a compensating correction factor may also be worked out by frequent checking of this reagent against standards.

In either of the two cases above, the checking or calibration could be performed as follows. The results of the Miller test could be compared with those of a Winkler test performed on two separate portions of the same clear water sample. Alternatively, a sample of water could be left exposed at a constant temperature and allowed to come to equilibrium with the oxygen in the air. The amount of dissolved oxygen which should be present in the sample can be obtained from tables to be found in texts. The actual results obtained with the Miller method could then be compared with this figure. The correction factor would be worked out as follows. If the Miller result read say, 9. where the Winkler or the theoretical result read 8. it would indicate that the Miller method was reading high. To correct for this, all subsequent Miller analysis done with these reagents should be multiplied by the factor 8/9 to reduce these results to a true reading. In the case where the Miller method gave low results of 7 when 8 was expected you would increase subsequent test results by a factor of 8/7. That is, the factor is the expected reading divided by the observed Miller reading.

Disadvantages of the Miller Method:

There are no modifications of this method to compensate for interferences. Good technique and rapid analysis must be used to avoid large errors. It is possible to remove suspended organic solids by the same methods used for the Winkler method. Where approximate results are desired, the method may be quite valuable. In any case, do not accept results without question. Always keep in mind that the results for both these methods may be seriously in error if good techniques are not employed.

LABORATORY DEMONSTRATION

VOLATILE ACID TEST

by

P. DIOSADY

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An Address To
The Ontario Water Resources Commission
Intermediate Sewage Works Operators' Course
Toronto, Ontario
March 8, 1962

LABORATORY DEMONSTRATION

VOLATILE ACID TEST

by

P. DIOSADY

INTRODUCTION

The volatile acid determination is considered to be one of the most suitable tests for controlling the anaerobic digestion process.

The main purpose of this process is to reduce the sludge volume, and this is achieved by the biological action of anaerobic bacteria. One group of these bacteria, the acid forming bacteria bring the insoluble organics into solution and break them down into smaller molecules, a part of which are volatile fatty acids.

Another group, the gas forming bacteria, further decompose these compounds in solution, producing gas. The decomposition of the organic material allows the particles now mainly inorganic, to settle, thus reducing the sludge volume.

In efficient digester operation the rate of decomposition by the acid forming bacteria is equal to the rate of introduction of sludge and the rate of the second group of reactions is also the same. Therefore, the composition of the solution remains practically constant.

Consequently, if one compound or group of compounds in the solution could be determined, this would give indirectly a measure of the digester operation. The easiest group to separate and estimate is the volatile acids. There are several methods proposed for this test. The one demonstrated, taken from Standard Methods, 11th Edition, page 422 is a simple, direct distillation method, which requires less time and equipment and provides sufficient accuracy for control purposes.

The method consists of the following three steps:

a) Separation of the supernatant by centrifuging.

b) Distillation of the volatile acids.

c) Titration of the distillate.

EQUIPMENT

1) Centrifuge, with head to carry four 50 ml tubes or 200 ml bottles
2) Distillation flask, 500 ml capacity with ground glass Joint. Pyrex.
3) Condenser: Friedrich, Liebig or Graham condensers with ground glass joints can be used. Pyrex or equivalent
4) Connecting adapter tube to fit both flask and condenser
5) Electric heater for 500 ml flask, with heat regulator
6) Titration unit: 50 ml burette with stand
7) Standard glassware (pipettes, erlenmeyer flasks, graduated cylinders, etc.)

REAGENTS

1) Sulphuric acid 1.1
2) Sodium hydroxide solution 0.1N
3) Phenolphthalein indicator solution, 0.5 gr in 100 ml ethyl alcohol
4) Barium chloride solution 5% in distilled water

PROCEDURE

Centrifuge 200 ml of sample for 5 min. Pour off and combine the supernatant liquors. Place 100 ml of the supernatant liquor in a 500 ml distillation flask. Add 100 ml water, 4-5 clay chips or similar material to prevent bumping, and 5 ml sulphuric acid. Mix thoroughly so that the acid does not remain on the bottom of the flask. Connect the flask to a condenser and adapter tube and distill at the rate of about 5 ml per minute. Collect 150 ml distillate in a 250 ml conical flask. Add 2-3 drops of Barium chloride solution. If a persistent white precipitate forms, which indicates that some of the sulphuric acid was distilled over, discard the solution and repeat the distillation at a slower rate. If there is no precipitate with barium chloride, titrate the distillate with 0.1N NaOH solution using phenolphthalein as an indicator. The end point is the first pink coloration which persists for a short time.

CALCULATION

mg/l volatile acids as acetic acid

$$= \frac{\text{ml NaOH 0.1N} \times 6000}{\text{ml sample} \times 0.7}$$

REMARKS

1) It is important to use a relatively clear supernatant for distillation not only because the volatile acids are in solution, but also because the suspended material can cause troubles in distillation:

   a) Solid organic materials in suspension may decompose on heating with acid to yield additional quantities of volatile acids.

   b) The suspended material may cause local overheating, and the sulphuric acid may distill over causing high results.

   c) Such overheating might cause splashing and foaming, spraying the solution through the condenser and causing high results.

2) An electric heater with regulator seems to be most easily controlled but flame burners can be used as long as great care is taken to avoid overheating.

3) Instead of the Friedrich condenser shown, Graham or Liebig condensers can be used. (We use the Friedrich-type because it requires less space, it is easy to handle and if necessary it can also be used as a reflux condenser in other work.)

4) This method is an 'empirical' one. That is, the recovery of the volatile acids using this method <u>is not complete</u> and may vary from 65 per cent to 80 per cent (average 70%). It is very important to follow the directions very accurately and to make all determinations in exactly the same way, in order to get comparable results. To compensate for the incomplete recovery a facter of 1/0.7 is used in the calculation.

5) The volatile acid mixture is not always the same, the main part of it is usually acetic acid. For this reason, and to allow a comparison of results, the volatile acids are expressed as acetic acid.

6) If the whole distillate is titrated, the aliquot (ml sample used) is 100, and the formula is simplified to:

   Volatile acids in mg/l = 85.712 x ml NaOH 0.1N used.

   (If 0.117N NaOH is used for titration, the titration result multiplied by 100 gives the ppm volatile acids.)

6. Generally a value of 200-600 ppm indicates a well functioning digester. However, each plant should make several determinations in order to find out which is the optimum acid concentration. An increase over this value would show a surplus in volatile acid production due to overloading or a malfunction. This can have several reasons (drop in temperature, change in the composition of the sludge, presence of elements or compounds hindering the action of the gas forming bacteria etc.) and the operator should intervene accordingly.

7. pH measurements are not used to control digester operation, since they do not accurately reflect variations in volatile acid content. This is due to the buffering action of dissolved salts, which are present in appreciable concentrations in digester liquors. These salts tend to 'soak up' excess volatile acids, with little change in pH.

MAINTENANCE OF ELECTRICAL EQUIPMENT

by

G. CROOKSTON

Construction Division

An Address To
The Ontario Water Resources Commission
Intermediate Sewage Works Operators' Course
Toronto, Ontario
March 8, 1962

MAINTENANCE OF ELECTRICAL EQUIPMENT

by

G. CROOKSTON

Construction Division

INTRODUCTION

In dealing with the subject of maintenance of electrical equipment I believe you will find it of interest to have an answer to that famous question "What is electricity". I will therefore present the definition that now applies since "Atom" has become a household word, namely -

Electricity

To understand the action of electricity and of electric currents most scientists assume the existence of electricity as the actual entity of which all matter is composed. The Electron theory assumes that the 98 or more different kinds of atoms, which constitute all known physical matter, are all composed of Electrons having a unit negative charge and Protons having a unit of positive charge. The negative Electrons are relatively free to move and normally circulate about the positive Proton which forms the nucleus of the atom. The atoms of the various elements differ from each other only in the number and grouping of their Electrons. Since all matter in the neutral or uncharged state has equal quantities of positive and negative electricity, it is necessary to provide some force to cause the Electrons to move in a definite direction. It is this motion of the Electron, which, if confined to definite closed paths makes it possible for electricity to be harnessed and put to work. Since the Electrons are normally distributed evenly throughout a substance, a force called electromotive force is required to detach them from the atoms and make them flow in a definite direction. This force is often called "potential" and the unit for measuring this electromotive force is our old friend the "volt". The higher the voltage the greater the number of Electrons which will be caused to flow. The two main sources of electromotive force are primary batteries and electric generators. When a potential or voltage of sufficient strength is applied to a substance it causes a <u>flow of Electrons</u>. <u>This flow of Electrons is called an electric current</u>. <u>The rate of this flow of current is measured in amperes</u>, an ampere being the rate of flow of electric current represented by the movement of a unit quantity of electricity per second.

You now have a definition for Electricity and a Volt and Ampere. The product of the latter two items form Watts and we will now conclude this flight into the realm of electricity and move on to another subject of vital importance when dealing with the maintenance of electrical equipment namely Safety.

Safety

There are certain hazards in the installation, maintenance and use of electric wiring and equipment, of which you are generally aware. This force is normally silent and gives no warning of danger in the manner of a hot steam pipe, high presure air or water or moving machinery. Investigation of low voltage electrical accidents shows that their cause is often due to a failure to understand the hazard present. In our case we may consider the low voltage range as approx. 50 to 550 volts. Research done on the effects of electric current on the human body indicate that the severity of shock is determined by the amount of current flow through the victim. A milliampere is one thousandth part of an ampere and 50 ma. passing through your body (near the heart or lungs) can cause instant death.

Handling of Electrical Circuits

(a) Consider all electrical circuits to be dangerous. Shut off the power when examining or making repairs on light or power circuits. If this is not possible all necessary precautions shall be taken to avoid contact with live conductors.

(b) Lock or block open the control devices, open disconnect switches or remove fuses before working on power circuits. After these precautions attach tags to the device worded "Workmen working on line". The tag should bear the workman's name and the time the work started. The tag should remain on the device until removed by the workman whose name appears on the tag. In event a workman leaves without removing the tag it should only be removed after a thorough investigation by the person in charge.

(c) It is suggested that when working on equipment remote from the point at which the control switch is located, that the conductors be grounded at a point between the control switch and the work.

(d) Make a complete check of the circuit before applying power, after repairs.

Grounding of Equipment

The frames of all electrical equipment such as motors, generators, conduits, switchboxes, transformers etc. should be properly grounded when installed. Any equipment not found to be properly grounded during a maintenance check should be reported and steps taken to have the situation corrected.

Operation of Switches and Removal of Fuses

Determine the operating condition of any circuit before opening or closing a switch controlling the circuit. Open or close switches completely in a firm positive manner. This will prevent possible arcing or heating as the blades make or break contact. Open switches before removing fuses. Use an approved fuse puller to remove fuses on circuits of 550 volts and less, whether or not a switch is provided. Remove fuses by breaking contact with the "hot side" first and replace fuses by making contact with the "cold side" first.

Working about Motors, Switchboards, etc.

(a) Apply insulating tape to any connections to motors etc. or other electrical apparatus rated 110 volts or more, made for temporary hook-ups. Before removing the tape, after the test, shut off the power and protect the control switch with a tag.

(b) Keep oil cans, dusters etc. away from moving machinery.

(c) Keep metal objects away from live parts of switchboards.

(d) Use only approved dusters, brushes or wipers provided with insulated handles for cleaning switchboard, switches or other electrical equipment. Wire brushes are not recommended!

(e) Power should be shut off for cleaning normally live or moving parts.

(f) Discharge capacitors (condensers) before handling or making any connections, even with the switches controlling the capacitors open. Capacitors will store a charge sufficient to cause damage. A shorting bar can be made from wire and an insulating handle.

In concluding this section on Safety, I would like to point out that electric shock causes cessation of breathing due to muscular contraction and that artificial respiration is a must for a person subjected to a shock. This should be started as soon as the victim has been cleared from the circuit and a doctor should be sent for immediately.

PREVENTIVE OR PRODUCTIVE MAINTENANCE

Now to Deal with Preventive or Productive Maintenance

General Maintenance Requirements

(a) The first requirement in a completely satisfactory maintenance program, strictly speaking, is good electrical apparatus properly installed. No one can do a good maintenance job on equipment that is either not suitable for the job or with equipment poorly installed. If such conditions exist they should be brought to the attention of the proper party and corrected rather than try to establish a maintenance program.

(b) The second requirement for a good maintenance program is persons who have a thorough knowledge of the equipment's operation and have the ability to be able to make thorough inspections and minor repairs. However no maintenance man is expected to be able to completely overhaul and renew any piece of equipment. The maintenance of some pieces of equipment should be left to the specialist and you should use your own judgment in this respect or in some cases this will be decided for you. Complicated instruments, control equipment and electronic devices should be reported to the Plant Operations Division for service requirements and the necessary maintenance program will be set up.

(c) The third requirement of a good maintenance program is the establishment of "preventive maintenance". That is an all-inclusive phrase for the continuing inspection of equipment, the report and recording of the condition of the equipment, and the repair of the equipment. This system has already been established by the Plant Operations Division and good records at the plant with the information forwarded properly to the Plant Operations Division keep the system functioning correctly. Please contact Mr. C. W. Perry of the Plant Operations Division for advice on this system.

Inspection Records

A variety of record systems exist today and the General Electric film to follow will show one of these in operation. There are five basic record cards, and these may be combined or not as required. The five cards are:

1. The equipment record
2. A cost record or repair record card
3. An inspection check list
4. A maintenance schedule of inspections
5. An inventory control

Routine Inspection

With regard to routine inspection, it is necessary to decide each case on its own particular merit. In general, advice is available from this office as to when particular pieces of equipment should be inspected and this is based on the manufacturers recommendations and the usage of the equipment.

UNDERSTANDING MAINTENANCE OF ELECTRICAL EQUIPMENT

Unlike other types of machinery electrical equipment can be found in every operating condition that is known. Motors operate at room temperature, at below freezing temperatures, at very high boiling temperatures. They operate in air, in explosive atmospheres, under water, and many other diverse applications. The electrical lines run overhead, on the walls, under the floors and almost anywhere where there is space to put them in.

Because of this high diversity of electrical apparatus many maintenance people have the mistaken attitude that electrical apparatus is different from other machinery and will operate under almost any conditions. It is a tribute to the manufacturers that this is a general belief but it is exactly the opposite of the truth. The electrical equipment can be damaged more easily by operating conditions than almost any other piece of equipment. Water, dust, heat, cold, humidity, lack of humidity, vibrations and countless other conditions can affect the proper operation of electrical apparatus. Because of this, there are four cardinal rules to follow in maintaining electrical apparatus. These are:

1. Keep it clean.
2. Keep it dry.
3. Keep it tight.
4. Keep it friction free.

For each specific type of electrical equipment other considerations will also apply but if any of these four cardinal rules are violated trouble with the equipment can be expected. Reviewing these rules we find the following:

Keep it Clean

The principal cause of electrical equipment failure is dirt, whether that dirt is an accumulation of day to day dust, whether it is metallic particles from a nearby machine, whether it is lint or powdered chemicals it will contaminate and cause failure to electrical equipment. Dirt built up on moving electrical parts can cause fouling which leads to slow operating and arcing and subsequent burning. On coils it can lead to actual short circuiting and failure. Almost always dirt will lead to heating because of the higher resistance and to the insulating effect of the dirt retaining heat within the equipment. Every schedule of inspections for electrical apparatus should include a systematic schedule of cleaning of the apparatus.

Keep it Dry

Electrical apparatus operates best in a dry atmosphere for several reasons. One is that humidity on copper and iron parts that are used in electrical equipment can cause corrosion or rust which leads to higher resistance and heating and eventual failure. Another is that moisture in itself can cause short-circuiting and immediate failure. Other liquids such as oil can cause deterioration of electrical parts such as coil insulation which also leads to eventual failure. A final reason is that moisture can increase dirt build-up on electrical parts which also leads to failure. Special enclosures are offered by all manufacturers for equipment that must operate in moist conditions.

Keep it Tight

Most electrical apparatus operates with high speed movements which in turn will evenutally cause wear of the moving parts and imbalance. This imbalance tends to create vibrations in the equipment and loosen vital connecting parts. Routine servicing of electrical equipment includes the checking for tightness of all connecting parts as a simple precautionary measure. The tightening of a screw in a starter, taking a fraction of a second, can prevent loss of hours trving to check eventual trouble should the connection be loosened and fail. In rotating equipment especially, vibration cannot only damage the electrical equipment itself, but can offer serious safety hazards should the imbalance created get out of hand and cause deterioration of the equipment itself. For this reason mechanical parts such as bearings should be checked to make certain that the proper support is being offered the rotating rotor or armature at all times.

Keep it Friction-Free

Electrical equipment that is operating properly runs with a minimum amount of friction. As we have indicated above dealing with dirt, added friction to electrical equipment movement can cause serious difficulties. A good electrical inspection man will always check electrical equipment to see that it is operating with an absolute minimum of friction and has a safe, smooth operating movement.

While these four cardinal rules must always be kept for electrical equipment, it is not always necessary in the case of failure of electrical apparatus that the cause be electrical. As all four of these rules indicate, a great many so-called electrical failures are actually mechanical in nature and have nothing whatsoever to do with the electrical construction of the machinery. The failure of mechanical ball bearings in a motor can lead to motor failure which is electrical but with a mechanical root. Friction in contacts can cause an eventual electrical failure but here again friction of two moving mechanical parts could easily have been prevented and electrical failure would never have occurred. Such things as an unbalanced or bent shaft, an obstruction in a ventilating system, a loose connection, a faulty alignment, an unlubricated bearing, can all cause electrical failure but all are mechanical in nature. The inspector of electrical equipment should be made highly aware of the importance of inspecting the mechanical portions of all electrical equipment.

HINTS ON ACTUAL ELECTRICAL MAINTENANCE

The following information merely suggests steps that may be taken on routine inspection of electrical equipment. Time does not permit providing extensive details on service procedure, however, on the assumption that you are familiar to some extent with the maintenance requirements of the apparatus described, we will offer the following suggestions: -

A-C INDUCTION MOTOR - SQUIRREL CAGE AND WOUND ROTOR

The modern induction motor, squirrel cage type is probably the most rugged rotating electrical apparatus ever developed. Maintenance requirements, outages and repair costs, therefore, depend, to a very great extent on the correctness of the application. However, the first principle of electrical maintenance - KEEP THE APPARATUS CLEAN AND DRY is fundamental. This implies periodic inspections, which are a very desirable check on operating conditions.

The following are some checks that may be made during routine inspection of an induction motor: -

Voltage and Frequency

For best results, induction motors should be operated at their normal voltage and frequency. Some variation from normal voltage and frequency is allowable: the voltage limits being approximately $\pm$ 10% from normal and the frequency limits not exceeding $\pm$ 5%. Voltage should be checked at the motor terminals (or at the starter) during periodic inspection.

Maintenance of Stator Windings

The stator or (stationary) windings appear to be so simple and rugged as to cause one to frequently overlook the necessity for certain maintenance procedure. However, a glance into the average motor repair shop will make it apparent that the induction motor stator is after all a vulnerable piece of equipment. Most of the work going on will be replacing or repairing stator windings. Stator troubles can usually be traced to one of the following causes;

Worn bearings--moisture--overloading--operating single phase and poor insulation. Over lubrication is a very serious fault with motor installations. Dust and dirt are usually contributing factors. Some forms of dust are highly conductive and contribute materially to insulation breakdown. The effect of dust on the motor temperature through restriction of ventilation is another reason for keeping the machine clean, either by periodically blowing it out with compressed air or by dismantling and cleaning. The compressed air must be dry and throttled down to a low pressure which will not endanger the insulation.

This paragraph suggests that a periodic check should be made for worn bearings, possible overloading (check with an ammeter), and condition of the insulation. If the latter is dirty, of course it should be cleaned.

Moisture

One of the most subtle enemies of motor insulation is moisture. Needless to say, motor insulation must be kept reasonably dry, although many applications make this practically impossible unless the motor is totally enclosed. If operated in a damp place, special moisture-resisting treatment may be given the windings. A new motor or one that has been taken out of operation and possibly stored in a damp location should not be placed in service again until it is thoroughly dry. The motor should be disconnected from the line and a megger reading taken of the windings, i.e. an insulation resistance measurement between the windings and ground (frame). In general the motor should not be placed in service unless the insulation resistance, measured at 500 volts, is not less than one megohm or a limiting value recommended by the manufacturer. On the periodic check the motor insulation resistance should be measured and noted.

Motor Windings

The life of a winding depends upon keeping it in its original or new condition as long as possible. In the new machine the winding is snug in the slots and the insulation is fresh and flexible being newly treated with varnish, and, therefore, resistant to the deteriorating effects of moisture and other foreign matter. This condition is best maintained by periodic cleaning, following at longer intervals by varnish and oven treatments. One condition which frequently hastens winding failure is movement of the coils due to vibration during operation. After insulation dries out it loses its flexibility and the mechanical stresses caused by starting, as well as the natural stresses in operation under load, will precipitate short circuits in the coils and possible failures from coil to ground, this usually at the point where the coil leaves the slot.

Maintenance of Rotor Windings

The rotors of wound rotor motors have many features in common with the stators; therefore, the same comments apply to the care of rotor windings as are given for the care of stator windings. However, the rotor introduces some additional problems because it is a rotating element.

Most wound rotors have a three-phase winding, and are, therefore, susceptible to trouble from single phase operation. The first symptom of an open-rotor circuit is lack of torque, with slowing down in speed, accompanied by a growling noise, or perhaps failure to start the load. The first place to look for an open secondary circuit is in the resistance bank or the control circuit external to the rotor. Short-circuiting the rotor circuit at the slip rings and then operating the motor will usually determine whether the trouble is in the control circuit or in the rotor itself.

Squirrel Cage Rotors

Squirrel cage rotors are more rugged and in general require less maintenance than wound rotors, but may also give trouble due to open circuits or high resistance points in the rotor circuit. The symptoms of such conditions are in general the same as with wound rotor motors, that is, slowing down under load and reduced starting torque.

Brazing broken bars or replacing bars should be done only by a competent person and this kind of work should be done in the manufacturer's shop.

With a die-cast rotor a repair cannot usually be effectively made and the rotor should be replaced if defective.

Air Gap

It has been stated that a small air gap is characteristic of the induction motor. Good maintenance procedure calls for periodically checking the air gap with a feeler gauge to insure against a worn bearing that might permit the rotor to rub the laminations. These measurements should be made on the shaft end of the motor.

On large machines it is desirable to keep a record of these checks. Four measurements should be taken approximately 90° apart, one of these points being the load side, i.e. the point on the rotor periphery which corresponds with the load side of the bearing. A comparison of the new measurements with those previously recorded will permit the early detection of bearing wear. A very slight rub will generate heat sufficient to destroy the coil insulation.

Without doubt, the polyphase induction motor is the simplest and most fool-proof piece of rotating electrical apparatus. The largest single cause of winding failures is probably due to the rotor rubbing the stator iron, usually because of worn bearings or complete failure of the bearings.

MODERN MOTOR STARTERS

Good maintenance of motor starters requires a systematic program of inspection. While a specific schedule depends to a large extent upon the conditions of the application a general statement can be made that inspections should be made frequently enough to prevent serious trouble. Experience will soon indicate any installations upon which the service is most severe and such installations will require your most frequent inspection.

The personnel doing the inspecting should be skilled in this equipment and should be prepared to make quick repairs if necessary. An adequate system of recording inspection results and repairs is necessary for motor starters.

There are a number of general points with which every inspector of electric motor controllers should be thoroughly familiar in order to do a proper job. These include:

(1) Do everything possible for the safety of personnel.

(2) Initial installations should be tested and proved satisfactory as soon as possible. Apparatus should be easily accessible for inspection and repairs.

(3) An adequate supply of correct renewal parts should be kept available.

(4) Starter enclosures should be chosen as suitable for the operating conditions.

(5) Keep motor starters clean and dry.

(6) Replace contacts that are worn very thin or badly burned and pitted. Repalce contacts by pairs. Maintain correct contact pressures.

(7) Contacts should be kept clean. Do not change contact shape by rough filing or grinding.

(8) Keep contacts and all connections tight.

(9) Do not oil contactor or relay bearings but keep these units clean and with no friction in the moving parts.

(10) Operate coils at rated voltage. Both overvoltage and undervoltage conditions are undesirable.

(11) Keep arc-rupturing parts in good condition and in the correct operating positions.

(12) Replace frayed and worn shunts.

(13) Keep all dashpots clean. Be sure oil dashpots have correct oil in them.

(14) Correct conditions that cause excessive temperatures. Measure the temperature if in doubt about overheating.

(15) Be alert for undesirable grounds on all circuits and eliminate them.

Since replacement of contacts is a common need for motor starters we will deal with this point from the above list.

Every time a controller contacts open or close they are subject to mechanical wear and electrical burning. The reason for this is that the contacts close with a rolling movement combined with a wiping action which although it insures a good contact and confines the arcing with resultant burning to the tips of the contacts, both conditions cause wearing of the contact materials. Contact parts therefore are items that may require considerable maintenance depending upon the operating conditions. The actual mechanical wear of contacts that operate every second may be more serious than the electrical burning caused by the arcing.

As contacts wear, the material in them gradually disappears because of both mechanical wear and electrical burning. During the wearing process the contact pressures decrease. This affects the current-carrying ability of the contacts and if allowed to go too low will cause overheating of the contacts. A small contact with suitable pressure will carry current with less heating than a large contact with little or no pressure. Reasonable provisions are made for the wearing of the contacts, when the original designs are made, but replacements will eventually be necessary. Manufacturers will furnish information on correct contact pressures for their devices. The contact pressures may be reduced either because of worn contacts or damaged contact springs. If contact springs have been overheated they may be unable to provide sufficient pressure because the material has been weakened by the overheating. Contact pressures should be checked and maintained within suitable limits.

Always replace both moving and stationary contacts.

Inasmuch as contacts operate in pairs, replacement should never be made of a single contact so that a new contact will operate in conjunction with an old contact. Because of the wearing of the contact surfaces, the probability of a mixture of old and new parts operating badly is very high. The few extra cents and minutes spent in replacing both contacts will repay itself many times over in operating life.

Contacts are generally made of copper or silver. Silver contacts are generally used on the small curre$nt-carrying contacts or relays, electrical interlocks, pushbuttons, thermostats, pressure switches and similar devices. The remainder of the contacts are made of copper and large contacts for heavy currents are almost always made of copper. Contacts should be kept clean. This is especially true of copper contacts because the discoloration that soon appears on clean copper is not a good electrical conductor. It therefore increases the contact resistance and is often the cause of serious heating of contacts. When contacts are renewed it is important to clean the new contact, if it is discolored, and the surface against which it is mounted.

The slight rubbing action and burning that occur during normal good operation will generally keep the contact surfaces clean enough for good service. Copper contacts that seldom open or close however, will readily accumulate the thin discolored surface that may cause heating. This is not true of silver contacts. The dense discoloration that soon appears on clean silver is a relatively good electrical conductor. It is not necessary to keep silver contacts clean except for good appearance sake.

When excessive currents are closed or opened, or when contact motion is sluggish, the contact surfaces may be severely burned. If this burning causes deep pits or craters or a very rough surface, both the stationary and moving contacts should be renewed.

However, it is not essential or even desirable to have contact surfaces entirely smooth. Contacts with surfaces comparable to very coarse sandpaper may be considered in good condition.

Contacts that are dirty or excessively rough should be cleaned and smoothed with a fine file, but care should be used to maintain the true surface shape or contour of the original contact. The designer has spent much time and effort to determine the best contact shape. Changing the original shape by careless filing will leave high points or edges that may overheat. Emery paper should not be used to clean contacts since it is an electrical conductor. Furthermore, some particles become imbedded in the contact surfaces and will cause unnecessary wear.

KEEP CONTACTS AND ALL CONNECTIONS TIGHT

Any loose electrical connection will eventually cause trouble. An open circuit or an unreliable one may cause much lost time and because they are very often difficult to find. And a loose connection can cause a poor contact of high resistance. The copper oxide or discoloration increases the resistance of the contact surface. The higher resistance causes more heating and the increased temperature causes more oxidation and higher resistance. The effect is always cumulative and the heating increases until the parts overheat, deteriorate or burn.

The bolts or fastening devices that hold contacts in place should always be tight. Normal expansion and contraction of metals due to temperature changes or excessive vibration will cause bolts or nuts to become loose. Frequent checking for loose contacts is therefore advisable.

WELDING OF CONTACTS

Very few contacts close without some bounce or rebound. This is due to the reaction of the contact springs as they are compressed to provide the final contact pressure. When the contacts bounce they separate. At this time the contacts are carrying current and even though the separation be very small, an arc is created. This arc, if severe, may cause sharp projections of burned or roughened contact surfaces to overheat, and may weld or "freeze" the contact surfaces together. Under such conditions the contacts may not open when next expected to do so. Other causes of contact welding are excessive currents when contacts close or open; insufficient contact pressure; sluggish operation either when closing or opening and momentary closure of contacts without much or any pressure applied.

DO NOT OIL CONTACTOR OR RELAY BEARINGS

Since the correct operation of the contactor depends primarily on the unit being completely clean and free from foreign material, oil or other lubricant of any kind should not be used on

the contact bearing. Therefore, contactor and relay bearings are designed to require no lubrication. If lubricated, the accumulation of oil and dirt may cause sluggish mechanical action that impairs the arc-rupturing qualities of the device or causes welding of the contacts. Except for bearings of master switches, drum controllers and similar units, no lubrication of controller parts is necessary.

FUSES

Other items that should have a regular inspection is fuses and fuse mountings. Fuses initially installed should be of the correct rating for the service involved and if so, then when a fuse blows, it should be replaced by one of the same rating. If a short circuit blew the fuse, make sure the line has been cleared and the cause of short circuit removed before installing a new fuse. If the circuit has been overloaded, obviously the overload will have to be relieved, before installing a new fuse. If the circuit has been overloaded, obviously the overload will have to be relieved, before installing a new fuse. If motor starting current or other harmless overload blew the fuse then the fuse did not have sufficient time lag. The answer is to use a dual element fuse of proper rating. For a motor circuit the dual element fuse (such as Fusetron) should be rated at full load motor current.

Now for some suggestions on maintenance of the fuses and fuse clips:

1. Do not insert the fuse in a live circuit because the resulting arc would create a burr on the fuse copper blade and prevent good contact in the clips. This is also a very hazardous procedure.

2. If the inside of the clips are not bright, clean them with emery cloth.

3. If the blades are not bright, clean them with emery cloth.

4. If the fuse can be easily inserted into spring clips and can be easily rotated after it is inserted due to insufficient contact pressure take the fuse out and draw the clips together.

5. If you want to test for a degree of contact, insert the fuse in the clips, then try to insert a narrow piece of thin paper between the clips and the fuse copper blade, from the top of the clip. If the paper can be inserted even for a short distance you are not getting sufficient contact. The clips must be pulled together making sure at the same time that the clips are straight.

6. If the spring clips have lost their spring they should be replaced or clip clamps should be snug and clean, otherwise, local heating will develop which will tend to blow the fuse prematurely.

We have covered a few of the most important items around the plants from an electrical standpoint and additional information is available for the asking. We now wish to present the film provided by the Canadian General Electric Company dealing with an effective maintenance program. No fixed rules can be followed to solve problems of maintenance but some fundamental principles can be applied in varying degrees to suit your operations. The C.G.E. company call this approach "Productive Maintenance" and they describe it as the application of men, tools and materials to protect capital investment and to reduce operational cost. With that brief introduction gentlemen, let us now look at the film.

INFILTRATION INTO SEWERS

by

A. W. SHATTUCK

Director, Construction Division

An Address To
The Ontario Water Resources Commission
Intermediate Sewage Works Operators' Course
Toronto, Ontario
March 8, 1962

INFILTRATION INTO SEWERS

by

A. W. SHATTUCK

Director, Construction Division

The problem of infiltration into sewers is a very complex one. There are so many factors involved and so many conditions to be met that no one could know all the answers. Certainly I do not claim to, however I propose to discuss the importance of limiting infiltration into sewers to a reasonable degree and to discuss some ways and means of accomplishing this. When I have finished I hope you will enter into a general discussion of this most important subject for I am sure many of you have experienced difficulties with infiltration and will have suggestions which will be helpful to others. The limiting of the entrance of ground water and surface water into sanitary sewers is becoming more and more important with the increase in the use of sewage treatment plants and because the cost of pumping and sewage treatment has been steadily increasing. Also we will see that undue infiltration can lead to the use of larger sewers than would otherwise be necessary and to the need for relief sewers.

A few years ago a questionnaire was sent to 500 cities and towns in northwestern United States. In questioning these cities it was stated that an infiltration of from 50 to 100% of the diameter of the sewer in inch gallons per hour per 100 ft. of sewer would be acceptable. Using the more common method of specifying infiltration, that is, in inch gallons per mile per day this would be from 634 to 1268 inch gallons per mile per 24 hours. I think you will agree that these limits for infiltration are rather high. One-third of the cities reported that infiltration exceeded the allowable and some by as much as 50 times. Exfiltration in one city showed leakage of 100 times the allowable infiltration rates. Many places appear to have infiltration of around 20% of total flow.

Let us have a look at the cost of sewage treatment and pumping. I have 1961 costs from our Plant Operations Division for six plants across Ontario taken at random having

design capacities ranging from 0.75 m.g.p.d. to 12.5 m.g.p.d. These plants are generally operating at from 50 to 75 per cent of capacity. It can be expected therefore that costs per million gallons will be reduced somewhat as they approach design capacity. The costs per million gallons range from $59.00 to $137.30. These costs are operating costs only and do not include capital costs. On the basis of average capital costs and paying off the debt in 30 years at 5% the capital cost per million gallons assuming plants are operated to capacity range from $178.20 per million gallon for a ¼ million gallon per day plant to $55.40 for a 10 million gallon per day plant. If the plant is operated at 70% capacity then these costs become $255.00 per million gallons for the ¼ m.g.p.d. plant and $79.20 per million gallons for the 10 m.g.p.d. plant.

Let us consider a sewerage system with a daily average flow of 1 million gallons and an infiltration rate of 20% and see what we are talking about in dollars and cents. On the basis of the figures just quoted we can estimate the cost of treatment as $100 per million gallons which would be $100 per day and the capital cost as about the same amount or a total cost of treatment of $200 per day, of which 20% or $40.00 is for treating infiltrated water. The present value of the $40.00 per day over a 30 year period at 5% is $225,000.00. Obviously this situation would probably never occur in practice since the plant would have to be enlarged long before the end of the 30 year period. It does, however, point out the fact that handling infiltrated water costs money. The $225,000.00 would probably pay several times over the difference in cost of building good sewers as compared to sewers that leak.

Besides the cost of treatment there are added costs of pumping. Sewage normally has to be pumped at least once and in many systems several times before reaching the plant. At 70% wire to water efficiency and at 1¢ per kw. hr. the cost of lifting one million gallons one foot is 5.4¢. In the case of our 1 million gallon per day sewerage system and assuming a lift of 50 ft. which is reasonable the present value of the cost of power for pumping the infiltrated water would be over $3,000 which, in itself, is an appreciable item.

Now let us see where this infiltrated water comes from. First it may come from cracked pipe, faulty joints in both the sewer and the service connection, from house services or it may enter through manholes and catch basins. I would now like to deal with each of these in turn.

Cracked pipe will normally be the result of using an incorrect class of pipe. With knowledge of the type of soil, the depth of the trench and the manner in which the sewer is going to be laid, a proper selection of pipe can be made. Most pipe suppliers put out bulletins recommending the proper class of pipe for the various conditions. The Ontario Concrete Pipe Association has a publication entitled "Recommended Practice for Selection and Installation of Concerte Pipe Conduits". It is much easier to specify a particular type of trench, bedding and backfill than it is to get it. How many of you have seen

specifications where the backfill is to be hand tamped to one foot over the pipe to 100% Proctor. I have never seen this done and I do not remember ever having seen a Proctor test run on compacted trench backfill. Similarly in the case of trench width - it is one thing to specify a certain width of trench at the top of the pipe and another thing to get it. We have all seen wide trenches to the very botton. If these are allowed then the pipe should be of a class which will withstand the added load. If there is any doubt about the class of pipe which should be used I would suggest the higher class.

Next is the matter of pipe joints. First, what are the pre-requisites of a good joint. They should, of course, be watertight to prevent infiltration and root penetration. They should have some flexibility since there will always be some movement when backfill is placed on the newly laid pipe. They should be durable and non-corrosive and last but not least they should be easy to make. It is one thing to make a watertight joint for a demonstration on a concrete floor and quite another thing to make a good joint when working under adverse conditions.

Years ago the most common type of sewer pipe was vitrified clay pipe with bell and spigot ends and the joints were made with mortar or a combination of oakum and mortar. Concrete sewer pipe was joined in a similar manner. Mortar, as we all know, shrinks on setting resulting in cracks which admit water. If the line is rigid a differential movement between two pipe lengths may result in a broken bell. When enphasis began to be placed on reducing infiltration, engineers looked for a substitute for the mortar joint. Asphaltic and coal tar base jointing compounds began to be used. These joints are made using oakum and pouring the hot compound in the same manner as we used to pour lead joints with cast iron pipe. Some engineers used mortar outside of the compound to keep it in place. Some excellent results were obtained using this type of joint. The quality of this joint depends to a large extent on the type of workmanship. A more recent trend is to a pre-formed joint usually using rubber. At first this seemed to be the answer to all our problems but I fear that in many cases it led us into a false sense of security. In oder to be watertight adequate gasket pressures must be maintained. To do this the pipe ends must be made to very accurate dimensions. You will realize the difficulty in making concrete pipe to tolerance of only about 1/16 of an inch. Also the pipe has to be perfectly round. Until recently most of the concrete pipe using a rubber joint had a fairly large slope on the tongue and groove, between which the rubber gasket is compressed. When these pipes are pushed together there is a tendency for the gasket to push the pipe apart. This tendency is more noticeable in the case of small pipe than in the case of large pipe. Even this pipe with the large slope to the tongue and groove using a rubber gasket could be watertight assuming the pipe is made to accurate tolerances and provinding the pipes are kept pushed together, in other words in compression. We all know how contractors like to lay pipe. They put a plank across the end of the last section made and push it into the next pipe using the shovel of the backhoe. If the gasket pressure is relieved there is apt to be a leak. If you are laying pipe

where there is a high water table and a fine running sand even a small leak will admit some sand and this, in turn, leads to settlement of the pipe, opening of the joint still further and before long you have a pipe failure, and this failure may, before long, involve a good many pipe.

Most concrete pipe suppliers are now going in for a revised type of joint which looks very promising. The slope of the tongue and groove are limited to about 3 or 3½ degrees. This in itself reduces the tendency of pipe to spring apart, once they are pushed together. The most common joint being recently developed is similar to the old Lock Joint type of joint only using concrete surfaces instead of the metal end rings. They refer to this as the recessed tongue type of joint. I have some pictures which I would like to show you in a few minutes which illustrate the various types of joints either in use or being developed. These revised joints look most promising and if suppliers can maintain their tolerances within satisfactory limits I believe they will have a product which will be very satisfactory.

Now a word about the various types of pipe materials. Vitrified clay was one of the first materials used and it has many good qualities. It has long life being resistant to most types of industrial wastes. Bell and spigot pipe using a poured joint has been more or less successful depending on workmanship. It is difficult however to maintain accurate dimensions and with the trend toward the use of rubber joints the vitrified clay suppliers have come out with a plain end pipe using a wrap around rubber band over the joint with two metal bands to hold it in place. We have used this pipe on a number of jobs and my own view is that it is not as easy to get a watertight sewer with this pipe as with some others. With very careful and experienced workmanship it probably will meet the normal standards of infiltration.

Concrete pipe is made in so many types that one of them is likely to be satisfactory for the most difficult installations. More will be said about the various types of concrete pipe and their joints when we show the pictures.

Asbestos-cement pipe with which you are all familiar uses a collar and two rubber rings for joining. The surfaces are machined so of course very accurate tolerances are maintained. This is the sole reason for the excellence of this joint. If laid with reasonable care the joints are bottle tight, the only leakage being at manholes.

Normally house services are installed as far as the property line at the time of installing the street sewer. Also it is usual to install junctions in the main sewer to serve vacant lots when constructing the sewer. It is essential that junctions be properly sealed and the ends of service connections capped at the property line. The service connection from the property line to the house is usually constructed by plumbers and

unless there is provision for proper inspection of these connections they are apt to be inferior and a source of infiltration. I think we have all seen where service connections were left at the property line with a discarded cement sack stuffed in the end instead of being properly capped.

Unless a municipality has a by-law prohibiting the connection of footing drainage to the sewer and enforces it, these drains are bound to be connected to the sanitary sewer since the only alternatives are first, a separate storm sewer connection or a sump pump in the basement to pump the water out to the street. Both of these alternatives are costly. There have been reports of where as much as 55,000 g.p.d. has entered a service from the footing drains of one house. This was during the first year after construction and where granular material was used for backfill and no special attention given to compaction. In this particular case the infiltration reduced to 2500 g.p.d. after five years. While this is admittedly an extreme case, it does stress the importance of prohibiting the admittance of tile drains into sanitary sewers. Even at the 2500 g.p.d. rate after five years this is some 6 times the flow which could be expected from an average house if sanitary sewage only is admitted. An 8 inch lateral at minimum grade will serve some 1000 homes. If weep tile drains are admitted the number of houses served by an 8 inch sewer will be materially reduced or because of overload basements may be flooded after rains or in the Spring.

Aside from the cost of treating this additional water and the possibility of overloading sewers, these drains are bound to carry silt into the sewers. This collects in sedimentation tanks and digesters resulting in costly operation of the treatment plant as well as excessive wear of pumps.

During construction of houses contractors often dewater their excavations after a heavy rain by letting this water into the service connection.

We laid trunk sewers in one municipality a couple of years ago and already the sewers are loaded to capacity. We do not have the whole story as yet but we are confident the main trouble is from foundation drainage. The only answer seems to be a by-law restricting the admittance of tile drains and adequate inspection of private service connections.

Even in municipalities which have separate sewer systems cases arise where it is inconvenient to connect all catch basins to storm sewers. There is a tendency to connect the odd catch basin to domestic sewers always as a temporary measure. It is much easier to connect these however, than to change them over to the storm sewer at a later date.

Care should also be taken to see that a minimum of surface water enters domestic sewers through manholes by seeing to it that manhole frames and covers fit properly.

Any time the flow, after rains, increases to several times the normal flow you can be sure water is entering by some of the means we have discussed.

Now in regard to specifications, it is the engineer's responsibility to see that the specifications are adequate taking into account the depth of sewer, the type of soil, the height of the water table, and the type of bedding. It is not enough to specify an infiltration rate which is not to be exceeded. If, for example, a pipe of too low strength is specified and later is found to be cracked and admitting ground water, I think there is considerable doubt whether the contractor could be held responsible. Similarly if a certain type of pipe is specified and the contractor lays it to the satisfaction of the engineer, can the contractor be held responsible? It should be remembered that a countractor is only responsible for his materials and workmanship and cannot be held responsible for design. The engineer should satisfy himself that the pipe specified will be adequate if properly laid then see to it that the specifications are strictly adhered to.

There is now no problem in specifying asbestos-cement sewer pipe. A.S.T.M. Specification C 428 covers pipe in classes from 1500 to 5000 and in sizes from 6 inch up to and including 36 inches.

The specifications for concrete sewer pipe used as sanitary sewers are normally A.S.T.M. C 14 dealing with unreinforced bell and spigot pipe and C 76 covering reinforced tongue and groove pipe. These specifications, while they cover such items as strength of concrete, reinforcing and dimensions, do not specify tolerances which are accurate enough for rubber gasket type joints. A more recent specification A.S.T.M. C 443 entitled "Joints for Circular Concrete Sewer and Culvert Pipe, Using Flexible Watertight Rubbertight Gaskets" gives tolerances which cannot be exceeded but is a very general specification and will therefore admit a wide variety of pipe joints. For example it would allow the tongue and groove to have a slope of as much as 12 degrees. While there have been many successful installations where pipe meeting this specification was used, there have also been a good many failures. After a failure has occurred it is most difficult to determine whether the fault was with the pipe or the pipe layer. In any case we have propared a supplementary specification which limits the types of joints to one of the revised types. As we get more experience witn these revised joints we undoubtedly will be revising our specifications.

The matter of inspection is a most important one in sewer construction. A good inspector should know his specifications thoroughly, he should know exactly what is required and how it should be accomplished. He should take nothing for granted. I believe most contractors are sincere in wanting to do a good job, after all reconstructing faulty sewers is added expense to them. However, they have their problems in getting and keeping good men and it is the man in the trench actually making the joints who will determine the success or failure of the line. You can be sure that if the

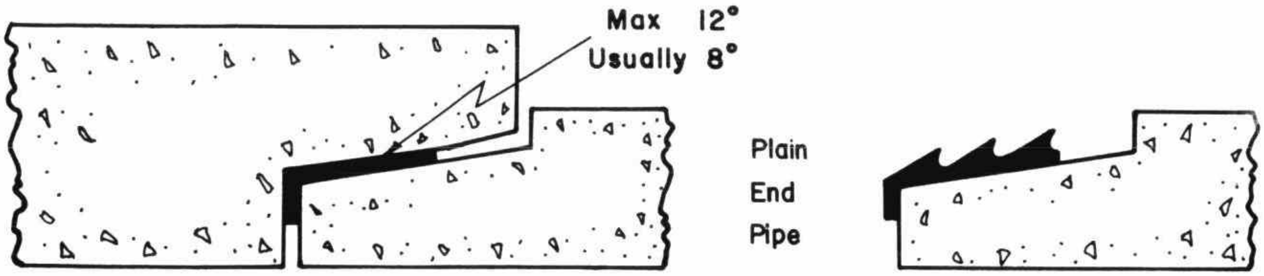
contractor has several jobs on at once he will have his best men on the job where the inspection is most rigid. It is the inspector's duty to see that the pipe layers know their job and do it. The engineer usually has authority to have any workman taken off the job and should do so if he is not doing his job properly.

Often the question arises as to whether or not there should be an inspector on every spread. Personally I would much prefer to see one inspector who is thoroughly competent looking after more than one spread than to have an inspector on each spread who is not competent. The mere fact that an engineer has a representative on each spread does not necessarily constitute inspection and often leads one into a false sense of security.

What should be the allowable rate of infiltration? There does not seem to be much uniformity in specifying allowable infiltration. We have specifications presented to us ranging all the way from bottle tight to 1500 inch gallons per mile per day. The tendency is to reduce the allowance. Keeping in mind that infiltration into manholes is included the allowed infiltration should be reasonable and a figure often quoted is 500 inch gallons per mile per day. It is not always possible to carry out an infiltration test. In some places the water table may at some time of the year be well above the sewer and at other times below it. Very often rather than specifying an infiltration rate, an exfiltration test is made. An exfiltration test cannot, of course, include the manholes unless the head is restricted to the height of the lowest manhole. A few years ago one recommended specification for infiltration allowance specified the maximum head on the pipe as 12 ft. The recent tendency is to use exfiltration tests and in some cases to test to a much higher head. Joint specification A.S.T.M. C 443 states that at a hydrostatic head of 25 ft. at the centre line of the pipe the exfiltration rate should not exceed 635 gallons. It should be kept in mind when specifying an exfiltration rate that with the same head on the pipe the exfiltration will be considerably more than the infiltration assuming the same head. I do not have figures for heads as high as 25 ft. but tests on the same section of pipe with a head of 8 ft. showed the infiltration was only two thirds as much as the exfiltration. The test referred to seems to indicate that at higher heads the infiltration rate would be a smaller percentage of the exfiltration rate. My own thought is that an exfiltration test with the head determined by the depth of the lowest manhole and an allowable exfiltration rate of some 400 to 500 inch gallons would be reasonable, keeping in mind the infiltration will only be about half the exfiltration.

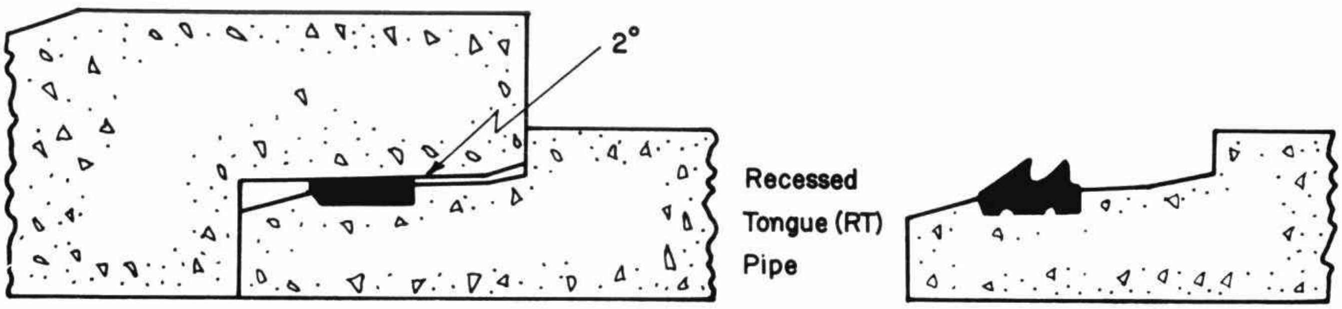
Even with carefully prepared specifications, adequate design and careful inspection, some faults are going to get by. The odd fault is not too serious in sewers larger than 24 inches since they can be repaired from the inside. In the case of smaller sewers however, leaks in sewers could be a source of serious trouble later on. The last year or so there has been

1



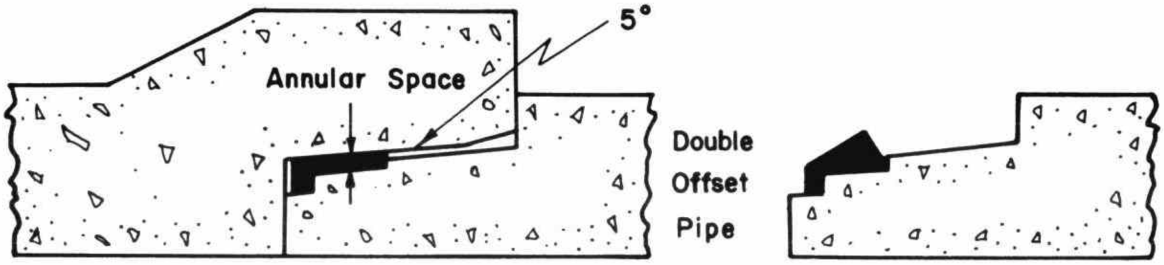
Max 12°
Usually 8°
Plain
End
Pipe

2



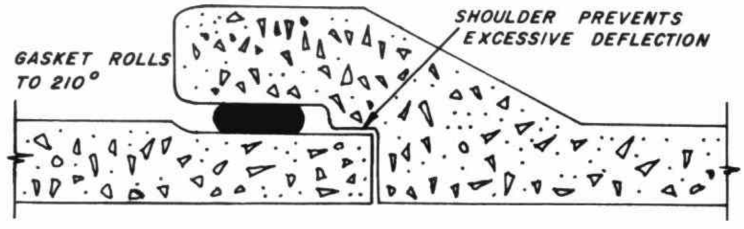
2°
Recessed
Tongue (RT)
Pipe

3



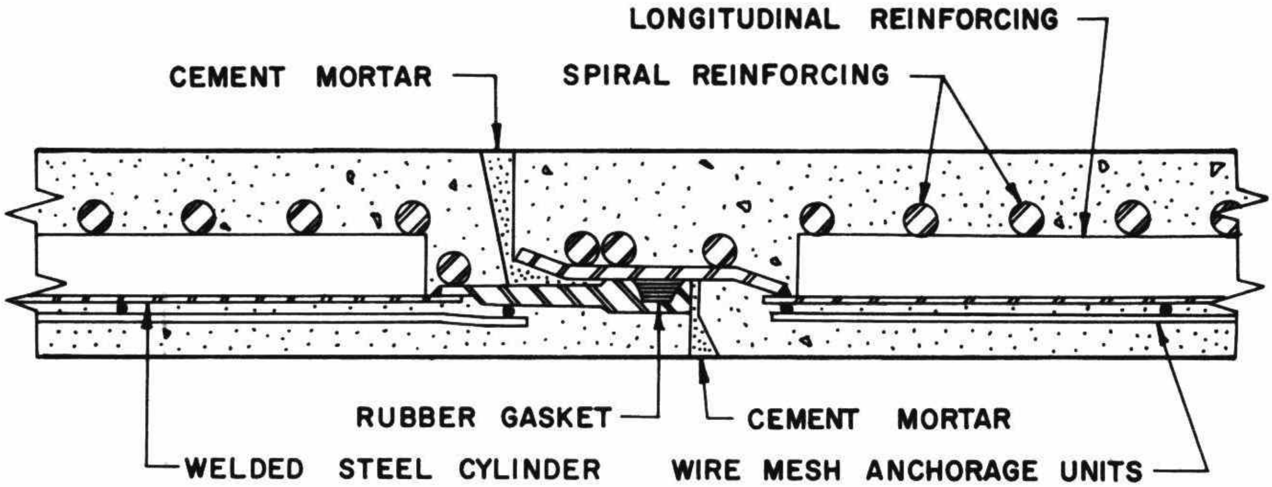
5°
Annular Space
Double
Offset
Pipe

4



SHOULDER PREVENTS
EXCESSIVE DEFLECTION
GASKET ROLLS
TO 210°

"HYPRESCON"
WOUND CYLINDER CONCRETE PIPE



5

COMPLETED JOINT

more and more interest in the use of pictures and closed circuit television for examining sewers after construction and prior to final acceptance. We have made use both of the camera which takes pictures every 5 or 6 feet and the closed circuit television. Neither of these give as good results as the firms doing this work indicate in their literature, however, we have had reasonably good success and propose to make more use of closed circuit television. We now include in our specifications a clause to the effect that we may have the sewers inspected by means of television and if faults are found the contractor is liable for the cost of the television inspection for the portions of the sewer which are faulty. We do not intend to have all sewers inspected in this way but if there is reasonable doubt as to the quality of the work we will do so. We feel the mere fact that this clause is in our specifications may result in better workmanship.

The following comments refer to pipe joints shown in Diagram 1 to 5 inclusive.

Diagram No. 1:

This joint is what is commonly referred to as the press seal joint. Notice the large slope on the tongue and groove. There is a tendency for the rubber gasket when compressed to push the pipe apart. Also in the case of differential movement between the two pipe ends the rubber could be subjected to the full weight of one of the pipes. I believe a good joint should be designed so that this could not happen. In other words concrete surfaces should come in contact so that the rubber gasket cannot be compressed more than 50% and yet maintain at least 25% compression on the gasket on the other side of the joint.

Diagram No. 2:

This joint is referred to as the recessed tongue joint. The gasket can be as shown or some suppliers prefer the use of an O-ring gasket. You will notice that when the joint has been completed the gasket is confined on four sides. If the tolerances are correct the gasket cannot be over-compressed and yet there will always be sufficient gasket pressure on the opposite side of the pipe.

Diagram No. 3:

This joint is referred to as the double offset joint. A joint similar to it is the single offset which does not have a second offset adjacent to the end of the tongue. While the angle in this particular joint is shown as 5° it would be preferable to have less than 5°. Some suppliers are making it with a 2° slope. When the gasket is installed it is confined on three sides.

Diagram No. 4:

This is normally referred to as the Roll-on type of joint and is very similar to the old Simplex joint used by Canadian Johns-Manville except that there is a bell on one end of the pipe rather than using two rubber gaskets and a collar. You will notice here again that the joint is so designed that the gasket cannot be over-compressed and yet there will always be sufficient gasket pressure. This gasket rolls into position so no lubricants are required.

Diagram No. 5:

This is a the old Lock-joint type of joint. The main difference between this and the joint shown in Diagram No. 2 is that metal end rings are used. While the diagram shows a pipe with a steel shell the identical joint is used for concrete pipe made to A.W.W.A. Specification C 302. This is an excellent joint but is expensive due to the use of the metal rings.

MAINTENANCE SCHEDULES IN SEWAGE

TREATMENT PLANTS

by

C. W. PERRY

Supervisor, Plant Operations

An Address To
The Ontario Water Resources Commission
Intermediate Sewage Works Operators' Course
Toronto, Ontario
March 8, 1962

MAINTENANCE SCHEDULES IN SEWAGE

TREATMENT PLANT

by

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Equipment maintenance is one of the keys to efficient operation of a treatment plant. The equipment that is designed or selected with extreme care by the Consulting Engineer or your engineering department, purchased under detailed specifications and operated by qualified men will not be efficient for long unless it is cared for. Such care is normally of two types:-

1. Breakdown maintenance and

2. Preventive maintenance.

Breakdown maintenance is the repair of equipment after failure and is usually of an emergency nature. Preventive maintenance, on the other hand is planned or scheduled maintenance of the 'fix it before it breaks' type, and is designed to eliminate or minimize breakdown maintenance.

The value of eliminating or nearly eliminating breakdown maintenance in the treatment plant can be easily appreciated. If treatment is to be continuous, the treatment plant can not let its major pieces of machinery run until they fail and then repair them unless it is supplied with an over abundance of standby equipment. As you all know, equipment has an uncanny and unfortunate habit of breaking down when it is needed most or when it is most awkward or difficult to repair.

Preventive maintenance, which by definition is scheduled and planned maintenance, is an alternative to an over abundance of standby equipment. It is not now, nor has it ever been economically feasible to provide 100% or more standby for all equipment required on peak flow days. Thus, by providing for the repair and servicing of equipment at periods of the year when service to the public is not so critical as at other times, scheduled maintenance can provide a reasonable assurance that breakdowns will not occur when the equipment is needed.

I am not saying that all breakdown maintenance can be completely eliminated. A reasonable objective is that of spending not more than 20% of the man-hours of equipment maintenance on breakdown maintenance. With some types of equipment, such as centrifugal pumps, breakdown maintenance can be almost completely eliminated if scheduled preventive maintenance is adequate.

To bring breakdown maintenance to a reasonable level, it is necessary to give your maintenance policy the same careful attention that is usually given to the operation of equipment.

The basic features of a sound maintenance policy are:-

1. Responsibility for maintenance must be clearly defined and in the hands of competent personnel.
2. A thorough knowledge of the equipment.
3. Proper tools, spare parts, test instruments and shop facilities for maintenance.
4. Preventive action must be planned and scheduled.
5. An adequate system of written records and reports must be used to permit control over the program.

Because the need for maintenance is something that usually does not show, it is important to make clear and definite assignment of responsibility. When maintenance is everybodies' business, equipment breakdown becomes no bodies' fault. An assignment of responsibility for maintenance should usually be made to one person who has the capacity, aptitude, and is allowed time for the job. The person should be given the help he needs as indicated by the type, amount of maintenance work required, and the size of the operation.

The fact that the maintenance personnel must be qualified and experienced in maintenance work can not be over emphasized. Knowledge of such important maintenance factors as what constitutes excessive vibration, when ball bearings should be replaced, when welding constitutes a safe repair, what a loose fit is, and similar points, is only acquired by experience, training, and education.

The knowledge of your equipment can be obtained from the instruction manuals issued by the manufactures of the equipment. These manuals, as distributed to you, are a result of many years accumulated experience on the part of the manufacturer. Experience gathered from all types of equipment application and tests both in the field and on the plant floor, and under all operating conditions.

An adequate supply of properly sized common shop tools are needed for efficient maintenance procedures. These tools should be of the best quality available. Cheap tools can not only damage equipment but will be more costly in the long run. Special tools for certain units should be obtained from the manufacturer. The tools themselves should be taken care of. The tools a mechanic uses and their condition is a good indicator of the calibre of the mechanic. Repair parts should be stocked according to the number of machines or equipment installed and the expected life of the equipment parts. The manufacturer can give you helpful suggestions on this.

The establishment of maintenance and inspection on a regularly scheduled basis is the initial step in setting up a preventive maintenance program which will assure continuity of

plant operation. Preventive maintenance refers to inspections and equipment care operations performed routinely according to the requirements of the equipment. For example, preventive maintenance schedules mean keeping equipment clean, in a state of good order and proper operation, free of excessive vibration, properly lubricated, and free of overloads and improper heating. Preventive maintenance requires a periodic check for wear and then the replacement or repair of parts before breakdown occurs. Such inspections often require the complete disassembling and reassembling of the equipment on a routine scheduled basis.

Preventive maintenance is successfully practiced when it is properly planned and performed according to a prearranged schedule. It requires the same attention and should be given the same importance as the transfer of sludge, holding digester temperatures or the cleaning of bar screens. Unless preventive maintenance is routinely performed and regarded as much a routine requirement of plant operation as are other plant operations, breakdown maintenance can not be avoided.

If for example, experience indicates it is desirable to blow out an electric motor on a chemical conveyor at least once a week, then this clean up operation must be performed routinely, without miss, if the performance life of the motor is to be preserved. To miss a week is to invite a second miss and a third, until breakdown finally tells you in a forcible, expensive way, how long a dirty motor will operate.

Again if a manufacturer has recommended the lubrication of a bearing every three months, with a particular grade oil or grease, experience has usually confirmed the reasonableness of this recommendation. Good judgement dictates that this lubrication should be scheduled routinely as recommended and not just when the operator happens to remember to do it, or with what ever lubricant is at hand.

To say that, unless a treatment plant follows a scheduled maintenance program, breakdowns and interruptions in service are inevitable may seem to be stretching the truth. But it can be said without any reservation that, unless preventive maintenance is scheduled and routinely performed <u>on time</u>, there is no assurance that such breakdowns will not occur. In other words, I am not prepared to gamble that if you have a scheduled maintenance program, you won't have a breakdown, but I will give you odds that you will have breakdowns without scheduled maintenance.

The planning of a preventive maintenance program therefore, requires that equipment items be studied to determine the maintenance operations required and the frequency of these operations. This is determined from an analysis of service conditions of the equipment, from your operating experiences, from others' operating experiences and from equipment manufacturers recommendations. Each equipment item or unit must be studied individually, for even similar pieces of equipment may have different care requirements because of location or service.

To carry out and administer a maintenance program effectively requires a minimum of records and forms, yet such records are very important to successful maintenance. First, the planned and scheduled portion of maintenance must be tabulated and calendered. Preventive maintenance operations are too numerous, even in the smallest plant, to depend upon the memory of one or more individuals. Such a practice is just as ill advised as depending on one or two old timers to remember where some of your man-holes are located after they have been covered over by the roads superintendent with 3 inches of black top. Then, too, maintenance planning is apt to be more thorough when the schedule is started in writing. Preventive maintenance records are also necessary to note when preventive maintenance operations were last performed, who was the maintenance mechanic, and what work was done. Finally, a record of the extent of unscheduled maintenance is also necessary. An analysis of these and their causes may indicate that they could be prevented if scheduled maintenance had been performed.

These record requirements can usually be conveniently accomplished with not more than two forms - one for preventive maintenance and the other for unscheduled or breakdown maintenance.

In the Commission, our Chief of Maintenance has developed a set of preventive maintenance forms, the use of which has been standardized through out all our projects either water or sewage treatment and applies to either large or small projects. It can be called a work reminder or a scheduled preventive maintenance form. Its first purpose is to list the preventive maintenance work required on each piece of equipment. Its second purpose is to list how often and when what work is to be done. The third purpose of the forms is to report preventive maintenance operations when completed.

With this simple form a very complete preventive maintenance program can be administered. An indication of how it is administered may be observed from the card.

1. Each card lists preventive maintenance for a definite piece of equipment.
2. Each card lists only the preventive maintenance work scheduled of a particular frequency and time.
3. There may be several cards for each piece of equipment if different work is required at different time periods. For a particular sludge pump, for instance, there could be a card listing the preventive maintenance work to be done weekly. Another card form listing the work to be done monthly, another card listing the work to be done quarterly, another card for semi - annual work and finally a card for work to be done on an annual basis. For purposes of simplification, we use different coloured cards for work to be done at different periods or intervals.

The first use of the form is a work reminder at the time the work is to be done. The cards are kept filed in a card file box along with cards pertaining to other equipment, according to the time the work listed on the card is next to be done. The coloured cards facilitates this work scheduling. The cards in this way become a work reminder file. After the work listed on the card has been completed the card is then moved forward in the file to the time when the scheduled work is next due.

These same cards can be used as work assignment forms. On a daily, or any other, basis chosen by the Chief Operator or superintendent, reference is made to the card file for work assignment. The cards are removed and are issued to the employee or employees who are to do the work.

Finally, these cards are used to record preventive maintenance work performed. Space is provided on the back of each card for installing for work performed and completed. The cards are then returned to the work reminder file in the appropriate section for the next scheduled date of preventive maintenance.

We have also developed a second form used primarily for breakdown or unscheduled maintenance and for new construction or improvements. This form has space for a description of work to be done and for a report as to possible causes of repair and materials used. An important use for these unscheduled maintenance reports is for information pointing to the need for a revision or addition in the scheduled maintenance assignments for the equipment or unit involved.

We have also instituted an additional card for each piece of equipment which has proved most useful in practice and all scheduled maintenance forms have been tied to this form for simplification. This form contains complete name plate and data of this unit referred to, together with, in case of a pump, the same information regarding the driving unit. The card also contains such pertinent information such as bearing size, part numbers, etc. We then have complete information regarding the unit in a concise easily available form.

A complete set of the cards mentioned above will be shown at the end of this lecture. Attention is drawn to several important features of the scheduled maintenance system and records as described.

1. The preventive maintenance program can be put into operation as soon as the preventive maintenance for any equipment is determined. Because the file is in a loose leaf form it is not necessary to wait until the entire plant's preventive maintenance requirements have been studied before beginning scheduled maintenance assignments.

2. Additions or revisions can be made merely by adding or exchanging cards in the card file. Other parts of the program are affected in no way.

3. Minimum of paper work is required. Very little clerical work - the bane of an operators or maintenance man's existence - is required to keep the program moving.

4. A historical record of previous maintenance work is always at hand and available for study.

In summary, where continuity of service and performance is of prime importance, as is in a sewage treatment plant, good maintenance is good management and good operation, and good maintenance is scheduled preventive maintenance that will cut breakdowns and unplanned equipment outages to a minimum, as well as keeping equipment operating at peak efficiency. No treatment plant is too small to install or institute a preventive maintenance program, and no community is rich enough to be able to afford not to take care of its equipment and get the most out of it.

Remember that you are not operating a plant that is handling a profit making product, you are only providing a service. A sewage treatment plant does not pay its own way and is a constant drain on the community's resources. It is therefore necessary for you to operate your plant as efficiently and as economically as is humanly possible and to use every known method of doing so.

HYDRAULIC FACTORS

IN SEWER DESIGN

by

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An Address To
The Ontario Water Resources Commission
Intermediate Sewage Works Operators' Course
Toronto, Ontario
March 9, 1962

HYDRAULIC FACTORS IN SEWER DESIGN

by

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Assistant District Engineer

INTRODUCTION

You are probably asking yourselves why there should be a paper on sewer design in a Course for treatment plant operators, since as operators you don't receive the sewage until after it has passed through the sewers. However, it is fact that sewer design is extremely important, and it is well worthwhile your taking a few moments to understand the difficulties of the designer and to know what happens when sewers are badly designed.

The sewage treatment plant design is based on an estimated strength and volume of sewage reaching the plant. Normally it is assumed that the sewage flows from the industry or dwelling unit through the sewer to the plant, and it reaches the latter in the same condition it started. Unfortunately, this may not be the case. Poor sewer design and construction can result in exceptionally long retention periods in the sewer and can allow considerable infiltration of ground water both of which cause considerable difficulty at the treatment plant. Long retention periods can result in stale, septic sewage reaching the plant and this puts a very heavy burden on the treatment units. Excessive infiltration of ground water can lead to hydraulic overloading of the plant long before the organic treatment capacity has been reached. Plant expansion might in this way, be required well before the size of the municipality warrants it.

In this paper, consideration is being given to sewers that are used for sanitary sewage only. It is true that many municipalities have some combined sewers which have been in the ground for many years, and those of you who have them, know what a nuisance they can be when it rains. Modern practice looks upon the use of combined sewers as a very poor substitute for separate sewers, and they are not recommended in Ontario. The design of storm sewers involves a considerable amount of calculation from rainfall and other data, and is beyond the scope of this lecture. Also, not included in this paper is a discussion on sewer materials. A discussion of the relative merits and disadvantages of sewer materials has been included in a paper on "Infiltration" by Mr. A. Shattuck, Director of Division of Construction, OWRC.

FACTORS IN DESIGN

When designing a sewer system, four major factors are taken into account which are:

(a)	Area to be served	-	present and future
(b)	Population to be served	-	present and future
(c)	Quantity of sewage anticipated	-	present and future
(d)	Type of sewage	-	present and future

(a) Area to be Served

The area that is to be serviced with sewers is first studied from the point-of-view of land usage, i.e., whether the land will be allocated to industrial, commercial or residential use or a combination of these. Maps of the area are obtained and the contours of the land prepared so that the sewers can be laid for gravity flow as much as possible. The rate of development is also studied since it is not always economical to size sewers to take the ultimate flow that could be collected. Lateral sewers within a development or subdivision are sized to take the ultimate flow based on full development, but trunk and interceptor sewers are more usually sized according to the anticipated or planned expansion for the next 25 to 50 years.

(b) Population

After the area to be served has been defined, the population which is now in the area and which is anticipated for the future must be known or estimated. The existing population, if any, can generally be assessed from census reports, density of housing, and types of industry where applicable. Registered subdivision plans can be used to obtain the future density of housing, but where these are not available estimates must be made. Residential districts usually have from 5 to 20 persons per acre depending on the lot sizes, land values etc., whereas tenement districts may have up to 100 or even more where multi-storey buildings are constructed. Manufacturing and industrial areas must be considered separately and based on the type of establishment anticipated.

(c) Quantity of Sewage

Many factors effect the volume of waste entering the sewers. In areas where the water supply is metered, a good estimate can be obtained from the water records. Amounts used for lawn-watering, car washing etc., should be deducted from the records to give a closer estimate. Industrial consumption and volumes of waste are much more difficult to estimate, and can usually only be determined from records or a study of similar operations in other locations.

Volumes of residentail wastes can vary widely according to the habits of the population and the housing conveniences installed. Large volumes of domestic waste can result from:

Laundry and dish-washing equipment.

Water-operated garbage grinders.

Water-cooled air-conditioning units.

Excessive use of showers.

Two other factors that can and often do drastically affect sewer calculations are infiltration and storm run-off where roof drains are connected to the domestic sewer. The latter problem is one that should not be allowed to occur, but is left as a relic of the past in many municipalities. Of course a number of municipalities have combined sewers and as we all know this presents one of the biggest head-aches of all to the plant operator.

Allowances for infiltration must be made in all sewer design. This factor varies widely according with the type of sewer pipe to be used, the type of ground, and the individual experience of the designer. Volumes in the range of 1,000 G.P.D. up to 1 M.G.P.D. per mile of pipe have been found but it is more usual to specify an amount equal to from 200 G.P.D. to 1500 G.P.D. per inch diameter per mile of pipe and then design the sewers according to this amount. Bad construction is usually the cause of excessive infiltration, due mostly to poor jointing, ill-fitting manhole covers, badly finished manholes, and abandoned house connections. Pipe settlement in bad ground can also seriously aggravate the problem.

For small areas, it is often sufficient to allow a total volume of 100 gallons per person per day (G.P.C.D.). Special buildings are treated separately, such as hospitals (up to 250 G.P.C.D.) motels (about 35 G.P.C.D.) and, school and recreational clubs (up to 15 G.P.C.D.).

(d) Type of Sewage

As far as sewer design is concerned, consideration of the type of sewage is usually taken only where strong industrial wastes are expected. Most sewer materials are able to withstand any wastes that would be acceptable at the treatment plant, and a number of municipalities have "Sewer-Use By-Laws" protecting them from the discharge of strong wastes and other undesirable materials to the sewer. Some municipalities, however, will accept strong wastes on payment of a surcharge. In this case it may be necessary to use specially resistant sewer materials for the sewers from the plant to a point where adequate dilution with other sewage can be assured.

Probably the most common chemical that affects sewers, causing excessive corrosion and pitting, is hydrogen sulphide. It is advisable to ensure that the discharge of wastes which would encourage the production of this gas in the sewers, is not allowed. It is worth remembering also that excessive retention of sewage, either in the sewers or in pumping stations before it is pumped to the sewer, also encourages the production of hydrogen sulphide as the sewage turns septic.

SEWER SIZING

Having studied the plans and decided on a layout that will permit the least amount of sewage pumping (preferably none at all), the next job is to decide the size of each sewer. Also, since the sewage is going to flow under the force of gravity only, it is necessary to calculate the gradient at which each sewer will be laid so that an adequate velocity can be assured. There are, therefore, three stages to the sizing of a sewer which are:

(a) What are the anticipated rates of flow?

(b) What size of sewer is required?

(c) At what gradient should the sewer be laid?

a) Expected Rates of Flow

In the previous section we discussed how to calculate the average daily flow from an area. This value is generally referred to as the Dry-Weather-Flow (D.W.F.). It is the average flow to be expected on dry days, when rain and ground-water infiltration will be at their minimum.

Of course, sewage flow, even from an entirely residential district, is not uniform, and the smaller the contributory area, the greater will be the fluctuations.

For small areas the following ratios are commonly used:

Maximum daily Flow - 2 D.W.F.

Maximum Hourly Flow - 3 D.W.F.

It is not uncommon however, to find laterals and sub-main sewers sized to take 4 D.W.F. for added safety. For very large areas it is not necessary to allow such large fluctuations, and a design based on 2-1/2 D.W.F. would probably be sufficient. It is interesting to calculate from your plant records this ratio each month or year (but of course not including storm run-off during wet days).

Unfortunately, it is not always economically practical to give consideration to minimum flow rates in sewer design. Particularly in small sewers these can reach almost zero. In trunk sewers however a minimum hourly flow of about 1/4 to 1/3 D.W.F. would be obtained and this figure would be considered from the point-of-view of ensuring sufficient velocity to prevent deposition of sewage solids (say 1 ft. per sec.). As long as a scouring velocity (2 ft./sec. or better) can be assured at least daily, no serious problems of septic or stale sewage should result.

As an example of the above discussion let us consider the flows expected from a 200 acre subdivision.

Population density - 5 lots/acre @ 4 persons/lot=20 persons/ac.

Volume of sewage - 100 gallons/person/day

D.W.F. = 100 x 20 x 200 = 400,000 G.P.D.

Maximum hourly flow expected - 4 D.W.F. = 4 x 400,000
= 1,600,000 G.P.D.

Minimum hourly flow expected - $\frac{1}{4}$ D.W.F. = $\frac{1}{4}$ x 400,000
= 100,000 G.P.D.

The maximum and minimum flows calculated above are of course rates of flow and not the actual flow expected during the hour. In other words a rate of 240,000 G.P.D. for an hour would produce a total flow of $\frac{240,000}{24}$ = 10,000 gallons in the hour.

For convenience, rates of sewage flow are usually expressed in either -- gallons per day (G.P.D.)
or -- cubic feet per second (c.f.s.)
The conversion factor is: 1 c.f.s. = 540,000 G.P.D. (approx.)

b) Size of Sewer

The selection of sewer size is based on the maximum rate of flow that is to be expected. The size cannot really be selected without the gradient, since both are dependent on each other, but as a general rule-of-thumb the following capacities are acceptables:

8"	sewer	-	380,000 G.P.D.
10"	sewer	-	590,000 G.P.D.
12"	sewer	-	750,000 G.P.D.
15"	sewer	-	1,300,000 G.P.D.
18"	sewer	-	1,900,000 G.P.D.
21"	sewer	-	2,700,000 G.P.D.
24"	sewer	-	3,400,000 G.P.D.

For the smaller sizes, the design flow would correspond to 4 x D.W.F., but for the larger sizes, it would probably be satisfactory to design on 2-1/2 x D.W.F. Sewers smaller than 8" in diameter are not recommended for municipal purposes.

c) Gradient of Sewer

The gradient of a sewer is selected so that a velocity of at least 2 ft./sec. can be assured at maximum daily flows, and if possible, a velocity of not less than 1 ft./sec. at minimum flows. A velocity of 2 ft./sec. once a day is generally sufficient to scour the invert of the sewer clean, but special conditions such as a heavy grease loading can result in a hold-up of solids even at this flow.

Selecting the best gradient is always a matter of economics. From the cost point-of-view it is preferable to keep the sewers as close to the ground surface as possible, and just deep enough to pick up basement drainage. On the other hand sewers laid to a flat gradient cause a lot more trouble through sluggish flows than do steep ones.

In the design of sewers it is a mistake to select a larger size pipe than that which is required in order to take advantage of a smaller allowable slope. For the same design flow, the velocity in the larger pipe on the smaller slope will be decidedly less than in the smaller proper size pipe on its required slope.

The following gradients are considered as absolute minimum values and should be exceeded if at all possible.

8"	sewer	-	Grade 4.0' per 1000 ft.
10"	sewer	-	Grade 2.8' per 1000 ft.
12"	sewer	-	Grade 2.2' per 1000 ft.
15"	sewer	-	Grade 1.5' per 1000 ft.
18"	sewer	-	Grade 1.2' per 1000 ft.
21"	sewer	-	Grade 1.0' per 1000 ft.
24"	sewer	-	Grade 0.8' per 1000 ft.

Nomo-graphs, tables or formulus can be used to obtain the flowing-fullcharacteristics of a circular sewer pipe of various standard sizes laid on certain slopes. For a given design discharge and desired minimum velocity, the required size and slope of the sewer can be found. Generally, however, the size falls between two standard sizes and the larger has to be chosen.

HOUSE SEWERS

House sewers should not be less than 6 inches in diameter preferably with a minimum slope of 1/4 inch per foot laid on a straight line and grade. Materials, joints and workmanship should be equal to those of the street sewer to minimize infiltration and root penetration.

Connections to the main street sewer should be made with "Y" or "T" branches. The "Y" or "T" branch should be installed with the branch turned about 45° from the horizontal, so back-flooding of the house connection will not occur when the lateral sewer flows full. The house connections for deep sewers may be made by means of a vertical pipe riser encased in concrete to prevent damage during back-filling.

All possible practical provision should be made for future connections in the original construction.

MANHOLES

Manholes are installed on sewer lines for ease of access for inspection and cleaning. On straight runs they are located at intervals of about 300' on small sewers, but at considerably greater intervals on larger lines. They are also installed at changes in direction, grade, or size of sewer, and junctions with other sewer laterals and mains.

Manholes are usually built of brick or concrete, and have been successfully constructed of pre-cast units. They should be at least 4 ft. in diameter to permit rodding operations to be carried out, and may be a good deal larger at junctions. They are placed to one side of the sewer so that a ledge can be left above the high flow mark, on which workers can stand. Covers are usually of cast iron and their design depends on the location of the manhole and the traffic loadings expected. The cover should be solid and fit snugly to prevent the entrance of storm water, grit and dirt into the sewer. Adequate ventilation can usually be obtained through the house connections.

Of greatest importance in the construction of manholes is the elimination of excessive infiltration. All joints must be tight and it is good practice to plaster the outsides of the structure.

Special manholes are built where necessary to facilitate the dropping of sewage from the level of a higher lateral, to a trunk sewer at a lower elevation. These are called drop manholes and when high flows and large drops are involved elaborate precautions may have to be taken to protect the bottom of the manhole from erosion damage.

INVERTED SIPHONS

The purpose of an inverted siphon or depressed sewer is to carry the flow under an obstruction such as a stream or depressed highway and to gain as much elevation as possible after the obstruction has been passed.

An inverted siphon is an arrangement whereby the sewage flows down the upstream side of the siphon and is then forced up the downstream leg by the pressure of the sewage in the upstream side. Calculations are based on the loss of head due to friction in the pipes. Depending on the size of the siphon a difference of levels across the obstruction of a few inches to a foot or so is the usual.

Siphons consist of two or more pipes, the smallest usually being laid in the lowest position and designed to accommodate the minimum flow. Velocities in the vicinity of 3 f.p.s. should be achieved to prevent blocking. Control gates should be provided at both ends so that any pipe can be taken out of service, and it is essential to have a drain at the low point for blow-off purposes.

LIFT STATIONS

It is occasionally necessary to install a pumping station in a sewer system so that the flow from one drainage area can be transferred to a second area. By the judicious use of small lift stations, particularly in flat terrain it is often possible to maintain satisfactory grades and avoid excessive excavation depths. Small package stations are not particularly expensive and may save tremendous excavation costs in areas where there is a lot of rock or other strata that would require blasting.

The design of the lift station is outside the scope of this paper but from the hydraulic aspects several points are worthy of mention.

The sewer (s) discharging to the wet well of the station is sized according to the principles already described, and the well is usually large enough and deep enough to avoid the sewer backing up. The discharge, or forcemain is designed in the same way as a water supply main according to friction loss principles, and must be capable of taking the maximum capacity of the pumps.

One design factor that is too often overlooked, and yet can become a real headache to the operator of the treatment plant, is the oversized pumping station. Unfortunately, it is often economically preferable to install a large capacity pump and cut down on the size of the pumping well. If a pump were to be designed to take the average maximum daily rate flow, then it would have to pump almost continuously on all average days to prevent the well from overflowing. This type of a station is actually the best of all as far as the treatment plant operator is concerned, since it gives him the least possible fluctuation in raw sewage flow. It may however require a very large wet well to provide sufficient storage for the peak flows, so that they can be pumped at periods of low flow.

The problem of fluctuating flows applies particularly to major pumping stations pumping directly, (or almost directly) to the treatment plant. A relatively large station at one extremity of the system will not affect the plant flow as seriously since the rate would become evened out by later gravity flow on route to the plant.

It is hoped that the forgoing comments have contributed in some way to the operators' understanding of the problems of the sewer designer. However, sewers are only as good as the contractor who lays them, and the man who checks the installation. The best and most efficient design of all, is not much use if it hasn't been built to specification. Badly laid sewers resulting in settlement, sluggish flows, cracked pipes and joints, and excessive infiltration can cause a lot of problems to the treatment plant operator which could have been avoided by good construction and checking procedures.

INDUSTRIAL WASTES II

by

R. H. MILLEST

Co-Supervisor, Industrial Wastes Branch

An Address To
The Ontario Water Resources Commission
Intermediate Sewage Works Operators' Course
Toronto, Ontario
March 9, 1962

INDUSTRIAL WASTES II

by

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There is little doubt any more of the need for providing for the treatment of industrial wastes in municipal sewage treatment plants. In many municipalities, where long-established industries now find themselves surrounded by urban development, combined treatment is the only practical way of handling industrial wastes. In some cases, industrial wastes are very difficult to treat alone, but, in mixture with sanitary wastes, are treated with comparative ease. Where a large industry is involved, combined treatment with a suitable cost-sharing can prove to be advantageous to both the municipality and the industry, as both the capital and operating costs of a joint plant can be lowered by combined treatment.

Most industrial wastes are amenable to conventional sewage treatment, but due to wide fluctuations in volume and characteristics, or the presence of toxic components, which may result from some industrial operations, control must be exercised at the source to ensure uninterrupted, satisfactory treatment plant operation.

As outlined in the first course given here last year, ten characteristics of industrial wastes should be considered:

1. Large flow volumes, or extreme fluctuations in flow.

2. High temperature.

3. Excessive concentration of floating or settleable solids.

4. High acidity or alkalinity.

5. High organic loading (as indicated by Biochemical oxygen demand).

6. Toxic substances (cyanide, chrome, copper, etc.).

7. Excessive quantities of greases and oils.

8. Explosive or combustible material which may present a hazard in the sewers or at the treatment plant.

9. Odours, or odour-forming substances.

10. Deficiency of essential nutrients, (or over-balance of one type of microbial food).

Limitations that should be imposed on the discharge of industrial wastes to sewers have been discussed previously. It is the purpose of this discussion to consider the sources and characteristics of the most commonly encountered industrial wastes, and how they can be controlled in keeping with the recommended permissible limits.

DAIRIES

Wastes from the dairy industry result mainly from receiving stations, bottling plants, creameries, condensing plants, and cheese manufacturing plants. They consist of the washings from cans, bottles, pipe lines, equipment such as separators and churns, the leakage from poorly made up joints in the pipe line, and the washing of floors and handling equipment.

As an indication of the potential waste loading from a milk handling or processing plant, it is noted that the 5-day BOD of whey, buttermilk, skim milk, and whole milk ranges from 32,000 to 102,500 parts per million, and the organic solids from 6.4% to 11.7%. It is obvious that any large-scale spills or dumping of such strong wastes can produce shock over-loading of a municipal plant, and therefore, milk plant operators should be aware of the effect of dumping and precautions taken to avoid it.

Barring any large-scale dumpings, milk-plant wastes are amenable to treatment with municipal sewage if secondary treatment is available. Since most of the milk solids are in dissolved or colloidal state, primary treatment is ineffective. In small municipalities the milk wastes may make up a disproportionately large part of the treatment plant loading, and it may be preferrable for the dairy to provide separate treatment in the form of spray irrigation or other accepted methods.

PLATING AND METAL FINISHING

The commonest discharges of toxic inorganic and metallic wastes orginate from the plating and metal finishing industries whose wastes contain varying concentrations of acid and alkali cleaners and degreasers, cyanide, chromium, nickel, copper, and zinc.

The industrial process producing these wastes requires the preparation of perfectly clean surfaces on which to plate out the desired metal finishes, and involves the use of relatively

large quantities of rinse waters to prevent the carry-over of one cleaning or plating solution to a subsequent step in the process. It has become fairly standard practice in recent years, to rinse first in a still-rinse (without over-flow), where most of the adhering chemicals can be removed and retained, then in a running or spray rinse to complete the washing. Without the use of still-rinses, the concentration of cyanide, for example, would probably be in the 30 to 40 parts per million range in the running rinse. The use of still rinse tanks (or counter-current washing) can reduce cyanide to less than 10 parts per million. Dilution with other process wastes prior to discharge to the sanitary sewer, will produce an effluent with considerably less than 10 parts per million metal ions (Cu, Ni, Zn, etc. will be similarly reduced).

Since there is little that the sewage treatment plant operator can do to adjust operations to treat toxic concentrations of cyanide and metal ions, it is obvious that adequate control must be maintained at the source. The literature indicates that 1 part per million of hydrogen cyanide retards the rate of formation of an activated sludge of high activity, and 2 parts per million definitely reduces the quality of the plant effluent. Trickling filters can be acclimatized to much higher concentrations, with as much as 25 to 30 parts per million of the cyanides of potassium, zinc, or cadmium, and up to 15 parts per million of cyanides of copper or nickel. This greater tolerance in trickling filters compared with the activated sludge process, is believed to be due to a specific growth on the filter media that converts these concentrations of cyanides to ammonium nitrate.

Although there is some doubt as to the amount of hexavalent chromium that can be handled in a sewage treatment plant, it appears that 2 parts per million impairs the activity of the activated sludge, and at 10 parts per million there is a definite deterioration in the quality of the final effluent. The main effect of chromium seems to be in the primary sludge. The rate of sludge digestion is reduced if the concentration of soluble chromium salts in the sludge is greater than 1 part per million. (This same effect is produced by about 200 parts per million of precipitated trivalent chromium).

Copper is mainly precipitated in the primary settling process, (a concentration of 60 parts per million in the raw sewage is reduced to less than 4 parts per million in the primary effluent). A concentration of 1 part per million adversely affects the activated sludge process, while a loading up to 2.9 parts per million does not appear to interfere with trickling filter operations.

The effect of copper on the results of BOD determinations should be kept in mind. The error produced is as shown in the following table:

Copper (ppm)	Reduction of BOD
0.01	5%
0.05	20%
0.40	42%

Less attention has been given to other toxic components of plating wastes, but it has been shown that the activated sludge process is affected by concentrations of 1 to 3 parts per million of nickel and as little as 0.5 parts per million of zinc. As with cyanide and copper, somewhat higher tolerances are noted for trickling filters.

In general, toxic wastes such as those from plating operations, appear to selectively eliminate and retard the growth of certain higher biological forms at relatively low concentrations. The remaining biological forms appear able to maintain a fairly effective sewage oxidizing floc up to some limiting concentration, usually in the range 5 to 100 parts per million. The general pattern of sewage oxidation in the presence of low concentrations of toxic substances, shows an initial lag in oxidation followed by a rapid oxidation up to some limit below that of a system free of toxicity. The initial lag is probably caused by the elimination of certain species, then as more tolerance species take hold, oxidation proceeds. The lower ultimate oxidation is probably due to the inability of the remaining species to oxidize the material to which the toxic ions are bound.

Treatment of plating wastes includes alkaline chlorination of cyanides, reduction and precipitation of chromium, precipitation of metal solids by pH adjustment, neutralization of acid and alkali cleaners, separation and removal of oil, etc. Each industry must be considered on its own merits, since the application of treatment and control will depend on the lay-out of the plant, schedule of operations, and other factors which will vary from plant to plant.

SLAUGHTERHOUSES AND PACKINGHOUSES

Wastes from the slaughterhouses and meat packing industry are characterized by high concentrations of BOD, suspended solids and grease. Wide fluctuations in both strength and quantity are encountered as processing operations change throughout the day. Surges of strong wastes result from batch dumping of hog-scald water, wet rendering waste water and wash-up. In evaluating the loading that can be expected from a packinghouse with a mixed kill, the following concentrations and quantities of waste per animal processed can be expected:

BOD	- 1,700 to 3,000 parts per million (OWRC surveys have shown the maximum to be as high as 8,000 ppm.)
Suspended Solids	- 700 to 1,400 parts per million

Ether solubles (fat)	-	200 to 500 parts per million
pH	-	5 to 8

Expressing the wastes in terms of hogs slaughtered and processed (1 cow equals 2.5 hogs):

Water consumption per hog unit	-	325 gallons
Waste flow per hog unit	-	250 gallons
BOD per hog unit	-	4.6 pounds
Suspended solids per hog unit	-	2.16 pounds

Since the wastes are amenable to treatment in a municipal sewage treatment plant, it becomes necessary only to limit the loading in keeping with the capacity of the municipal plant to provide suitable treatment of packinghouse wastes. Segregation and separate disposal (use) of blood and paunch manure should be mandatory. All remaining wastes, except cooling water should be blended, given primary treatment at the packinghouse, and discharged to the municipal system at a controlled rate. Because of the tendency to decompose rapidly, waste holding and blending facilities should provide for continuous mechanical or air agitation, and the effluent to the municipal sewer should be chlorinated to prevent anaerobic decomposition in the sewers.

It seems almost certain that nothing short of secondary treatment at the packinghouse would produce an effluent (even) approaching the strength of normal sanitary sewage, and therefore there would be little hope of having the industry comply with sewer ordinances as discussed previously.

Treatment at a municipal plant is quite feasible, however if in-plant controls are maintained to include control of wastes by rigid housekeeping, careful removal of blood and paunch manure, primary sedimentation with sludge and grease removal, and controlled rate of discharge to produce a uniform feed to the municipal plant. Treatment plant loading will be high, but as long as the required capacity is available, and shock or peak loadings are suitably controlled, no serious problems should exist.

POULTRY KILLING AND DRESSING

The processing of poultry for sale has become an important part of the meat industry during the past few years, and in most cases, the plants are located in municipalities with the wastes being discharged to the sanitary sewers.

Poultry processing operations and the waste produced from them are not unlike meat packing operations. Various steps in processing which produce wastes include killing, bleeding, scalding, picking, eviscerating, washing and chilling. Operations are usually on an 8 to 10 hour day, followed by several hours of clean-up during which solids are removed and all equipment and tanks are drained and washed. It has been estimated that 25% of the daily water usage occurs during the clean-up period.

During the processing operation, blood is usually collected from the killing area for by-product recovery or separate disposal or it may be permitted to congeal on the killing area floor and removed as a semi-solid once or twice a day. If blood is not recovered but is included in the waste, it may account for as much as 40% of the total BOD load from the processing plant.

Feathers always seem to be a problem in the treatment of poultry processing wastes. The feathers removed in the picking operation fall to a flume and are conveyed by water to a mechanical screen which removes them. A portion of the screen water is recirculated and used to assist in conveying the feathers to the feather screen. Feather removal from the waste water can be virtually complete if the proper screen is selected and given adequate supervision and maintenance.

Like packinghouse wastes, poultry processing wastes are amenable to treatment in a municipal treatment plant providing care is taken to remove coarse solids, feathers, blood, and fats and grease. In general, the BOD of the undiluted waste prior to discharge to the municipal sewer is in the range of 300 to 1,000 parts per million, with an average of about 650 parts per million, and suspended solids in the range of 250 to 500 parts per million, with an average of about 390 parts per million.

The most important single item to ensure satisfactory poultry process waste treatment is a fool-proof screening system which prevents large solids from reaching the sewer. Mechanical rotary screens for removing feathers and offal do a satisfactory job if adequately attended, operated and maintained. A fixed half-inch bar screen at the processing plant is recommended to prevent the flushing of troublesome solids down the sewer to the sewage treatment plant.

TANNERIES

The processing of hides in the manufacture of leather involves a number of separate operations which produce wastes with widely varying characteristics. The most important wastes are: (1) wash water from the green hides (high in ammonia); (2) alkaline wastes from the liming vats and dehairing machine; (3) wash water from flushing and draining floors; (4) spent tan liquor (acid) and rinse water from vats; and (5) drainage from fleshing machines (fleshings are collected and sold).

Because of the sequence of the tanning processes, the sewered wastes produce a wide fluctuation in pH as first alkaline and then acid wastes are dumped. However, preliminary screening to remove hair, fleshings and hide trimmings, mixing of the individual wastes to take advantage of neutralization of one with the other, followed by settling and sludge removal, will produce an effluent that can be treated in a municipal plant. After sedimentation, the tannery effluent should be held as briefly as possible to prevent the development of anaerobic conditions. For this reason, retention facilities at the

tannery should be kept to a minimum. Where sewage treatment plant capacity is adequate to handle the average tannery waste loading operating difficulties arise mainly because of fluctuations in waste strength or pH due to improper mixing, or to the development of septic conditions in the municipal sewers.

TEXTILE INDUSTRY

The many operations involved in textile preparation and finishing give rise to a variety of wastes with widely differing strength and composition. Flow volumes are relatively high as large quantities of water are used in rinsing following treatment in the various steps of processing. Most of the operations are carried out batch-wise, and it is the wide fluctuations in volume, strength and characteristics of the wastes that give rise to treatment plant operating difficulties.

As is the case in treating many other industrial wastes, the mixed wastes from a textile plant generally lend themselves to treatment in a municipal plant, provided capacity is available. Fine screening, to remove fibre or hair, followed by equalization of neutralization and regulation of dumping rates is usually all the preliminary treatment required. With the proper levelling out of waste flows, the strength of the waste is generally in keeping with the levels of BOD and suspended solids in sanitary waste. The colour of dye wastes are usually diluted enough so that the municipal plant can completely remove them in secondary treatment.

BREWERIES AND DISTILLERIES

Wastes from the brewing and distilling industries have long been a source of difficulty in treatment plant operations. Wide fluctuations in both strength and characteristics have been difficult to control, and it has been only fairly recently that adequate pre-treatment steps have been proposed or undertaken.

The discharge of spent grains or hops to the municipal sewer should be rigidly controlled. Spent grains are marketed profitably in either the wet or dried form and there is no reason why removal should not be quite complete.

BOD loading varies throughout the process, but holding and blending followed by controlled discharge of the wastes can produce an effluent which can be handled without difficulty. Brewery wastes are deficient in nitrogen and phosphorous, and addition of suitable nutrient at the treatment plant may be necessary to ensure satisfactory secondary treatment.

Variation of pH is of major importance. Strong alkaline wastes from washing and cleaning of equipment are dumped batch-wise, and, unless they are suitably held and mixed with other wastes, can completely upset the treatment process. Acid wastes from regenerating water conditioning facilities are also a source of trouble.

GENERAL

Looking at industrial wastes in general, most of those encountered in the treatment of municipal plants are readily treated with only moderate pre-treatment.

Screening is the simplest form of pre-treatment, and is particularly applicable to such wastes as those from vegetable canneries, tanneries, slaughterhouses and paper mills. Sedimentation with or without chemicals would have wider application in pre-treatment if it were not for the problem of sludge dewatering, and disposal. Centrifuging and filtering can overcome this drawback, but only at considerable capital outlay for equipment. Excessive oils and greases can be recovered efficiently in flotation devices varying from simple grease traps to elaborate devices employing the dissolved air flotation principle.

Controlled discharge becomes increasingly important as the industrial load becomes a significant portion of the total waste load. Maximum efficiency in treatment should be accomplished by keeping the influent mixture as uniform in composition and volume as practicable.

Increased use of chemicals for partial or complete treatment of industrial wastes may be expected. Applications range from simple neutralization to reduction of toxic chromium wastes with sulphur dioxide and alkaline chlorination to destroy cyanide. Coagulants may be justified for seasonal operations and to augment plain sedimentation where dewatering of sludge is feasible.

Above all, the sewage treatment process is a physical-biological process, and maximum efficiency is obtained when the inflowing sewage is of uniform quality and consists only of those components which lend themselves to treatment. Adjustment of industrial waste discharges should therefore be required to this end.

Since most industrial wastes differ in composition from "normal" sanitary wastes, the question of nutrients should usually be considered. The removal of organic pollutants from waste waters is accomplished primarily by bacteria and rapid removal depends upon unrestricted reproduction or growth of the bacteria by using a minimum amount of organic pollutant for energy, it follows that a fairly uniform rate of feed or nutrients of the right kind is necessary. For example, three industrial wastes that are known to be deficient in nitrogen and phosphorous, such as cotton kiering waste, rag rope pulping wastes, and brewery wastes, are not successfully treated without the addition of nutrients. From 5 to 7 pounds of nitrogen per 100 pounds of 5 day BOD and approximately 1 pound of phosphorous per 100 pounds of 5 day BOD are necessary in the activated sludge process. Where these wastes are known to be received in a treatment plant in appreciable quantities, nitrogen and phosphorous analyses should be made at each stage of the treatment process to determine if a deficiency exists.

The trend towards combined treatment is growing, and in conjunction with this so has the policy of asking the manufacturer to install, where necessary, some form of pre-treatment either physical or chemical, for treatment of the industrial wastes at the factory. Significant reductions in BOD, suspended solids, or harmful characteristics or constituents of specific wastes can be obtained. The degree of treatment that can be provided for any industrial waste depends on the ability of the treatment plant to remove and stabilize the waste product. If the treatment plant has the capacity to handle the wastes from a certain industry or group of industries, assuming reasonable precautions are observed at the industries, and a uniform quality (and quantity) is maintained, it may be in the best interest of both the industries and the municipality to provide the necessary treatment at the municipal plant with some form of surcharge for the service. Industry would probably quickly evaluate private -vs- municipal treatment, and arrange for whichever treatment proved to be to their advantage.

WASTE STABILIZATION PONDS

by

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An Address To
The Ontario Water Resources Commission
Intermediate Sewage Works Operators' Course
Toronto, Ontario
March 9, 1962

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INTRODUCTION

There has been no single method of waste treatment which has evoked as much interest in the past forty years as the waste stabilization pond, oxidation pond, sewage lagoon or whatever you prefer to call them. In fact, they are probably one of the oldest man-made sewage treatment devices.

Until some ten years ago, few people had ever heard of waste stabilization ponds for the treatment of sewage, although a few had been used for this purpose in North Dakota, Texas and a few other states. The United States Public Health Service now reports some six hundred installations serving thirty-nine states throughout the United States. In Alberta, out of 187 communities served by a sewage works system, 114 of these operate some type of ponding as a means of sewage treatment. Ontario, at present, boasts only ten municipal lagoon installations in operation but approximately thirty to forty such projects are in various stages of development. In addition to municipal installations, waste stabilization ponds have been used by schools, motels, restaurants, trailer parks and industries such as milk processing plants, meat packing plants, potato processing plants and canneries. Judged by the experiences in this method of treatment, these facilities have functioned with a minimum of difficulty and complaint other than the usual problems associated with placing a new sewage treatment plant in operation. Thus, it would appear that the use of stabilization ponds for treating raw domestic sewage is beyond the trial stage, at least in the United States and the western provinces and efforts are now being directed to refine the design criteria so that their application may be extended into other areas resulting in a corresponding reduction in sewage treatment costs.

In Ontario on the other hand, the experiences in the operation of this type of facility is somewhat limited at the present time and it will be necessary to evaluate the effectiveness of treatment over the next few years before definite conclusions can be reached regarding design criteria, operation and treatment efficiency.

MECHANISM OF STABILIZATION

A waste stabilization pond is a shallow excavation designed and constructed to receive raw or pretreated domestic sewage and some organic industrial wastes in which stabilization is accomplished by several natural self-purification "phenomena".

The purification process in the lagoon is dependent upon the combined action of wind, sunlight, temperature, sedimentation, bacteria and algae. A portion of the solids in the raw sewage settles immediately after the sewage enters the pond while the remainder are dispersed by wind action in the overlying water.

The bacteria, which is naturally present in the sewage and the soil, feed on the organic matter and by digestion convert it into substances which may be discharged into the receiving stream without creating objection or a depletion of oxygen. The abundant supply of soluble nutrients which include carbon dioxide, ammonia, nitrogen and other chemical substances released by the bacteria provide an ideal source of food supply for the algae which grow prolifically near the surface of the pond. The algae is dependent upon sunlight for their activity and release large quantities of oxygen except during periods when the sunlight is excluded. The oxygen in turn is utilized by the bacteria and is in fact absolutely essential to their activity.

Ice cover is a barrier to both light and wind. When snow is present, light penetration is further impaired. Consequently, stabilization ponds become anaerobic soon after ice formation. Accompanying low temperatures also slows down the bacterial action. At the same time, salts and other sewage constituents become more concentrated under the ice of non-overflowing installations, not only from the continual application of raw sewage, but also by exclusion from ice in the process of freezing. Therefore, melting ice can be expected to provide good quality dilution water in the spring.

Experience of the several states and provinces where lagoons have been used extensively indicate that the degree of treatment provided by waste stabilization ponds is generally equivalent to that provided by trickling filter and activated sludge type plants. The reduction in pollution as measured by B.O.D. and suspended solids is commonly in the range of 85 to 95 per cent. Reductions in coliform organisms of more than 99 per cent have been reported. No practical method is known today for the disinfection of the lagoon effluent to be discharged into the receiving stream. The high concentration of algae makes disinfection somewhat impractical. It is, therefore, essential that the pond be operated so that no overflow occurs when chlorination of a plant effluent would normally be required to protect the use of the natural waters. This can readily be accomplished by storage in the same manner in winter.

As mentioned previously, cold weather and diminished solar activity in the late fall and early winter reduces the algae and bacterial activity. When the lagoon is ice covered, this activity ceases although some decomposition continues under the ice in the absence of oxygen or under anaerobic conditions. When the ice melts, the gases generated by the anaerobic decomposition process are released to the atmosphere. These gases may become quite offensive, creating odour nuisances in the surrounding area and may persist for periods varying from a few days to several weeks. This is the most critical time of the year in terms of treatment efficiency, difficulty of management and aesthetics.

In some applications, it is necessary to provide storage during the winter months when the lagoon provides a low degree of treatment. This may be accomplished by lowering the water level in the fall of the year when the degree of treatment is high and the streams are in good condition to receive the treated effluent.

DESIGN CRITERIA AND CONSTRUCTION DETAILS

Location:

The location of a waste stabilization pond should be based on several factors, namely, cost of land, elevation, prevailing winds, topography, soil characteristics, and proximity and direction of flow in ground water aquifers. The site should be preferably located on the leeward side (downwind) of residential developments in relation to the generally prevailing wind direction and at least 1500 feet from the nearest habitation. In the majority of installations, the elevation of the site has been such that pumping of the sewage flow has been necessary. Soil characteristics should be such so as to produce a minimum percolation into the ground. A seepage rate of over 3/16ths of an inch per day may require artificial sealing of the lagoon bottom. Gravel and limestone formations should be avoided.

Surface Area:

Three basic types of operation should be considered in incorporating area into the design:

(1) - Total retention (no overflow),

(2) - Intermittent drawdown (based on receiving stream requirements),

(3) - Continuous overflow.

Fundamental to each type is the maximum loading of the flow and strength equivalent of 100 persons per acre or 20 lbs. of B.O.D. per acre per day. For complete retention, the flow of applied sewage per acre must balance the natural water loss.

Considerable data are available on experiences in other states and provinces where higher loadings were utilized with success. However, it is felt that more experience is necessary in this province under the climatic conditions which prevail before the design factor can be refined.

Shape:

The overall shape of a waste stabilization pond is not particularly important except as it relates to the surrounding topography. Round, square, or rectangular shapes may be used but care should be taken to ensure that coves, islands or peninsulas are prohibited since they may interfere with circulation and develop local nuisance conditions. Where rectangular shapes are used, the length should not exceed three times the width.

Depth:

The optimum depth varies with the season of the year. During the winter season, when the ice thickness may vary from less than one foot to well over three feet, a total depth of five feet is desirable. In the early spring, immediately after the ice has been removed, a shallow depth of about 2.5 feet would encourage rapid algae growth. In the late spring, summer, and early fall, an intermediate depth of 3 to 4 feet should produce more uniform temperature conditions. Provision should be made in the design of the overflow device for variable level control between a maximum depth of 5 feet to a minimum of 2 feet. At least 3 feet of freeboard should be provided.

Bottom:

The pond bottom should be graded level for uniformity of water level control. There should be no obstruction to circulation such as raised inlet pipes, debris, or weed growth when the initial application of sewage is made. Bentonite, asphaltic coating or other suitable material may be used to control the rate of percolation.

Dykes:

Compacted embankments of impervious materials should be constructed with a minimum embankment top width of 8 feet. Maximum embankment slopes should not be steeper than 3 horizontal to 1 vertical for both inner and outer walls. The mimimum freeboard should be 3 feet plus frost heave. Embankments should be seeded except below the water line. Additional protection for embankments such as rip-rap may be necessary and should be incorporated at the time of initial construction at least along the banks subject to wave action from the prevailing winds.

Inlet Structure:

The influent line into single-cell ponds should be essentially center discharging. Influent lines into the primary section of multiple cell ponds should also be center discharging but this does not apply to those cells following the primary cell in series operation.

Either upward or horizontal discharging influent lines may be used where the sewage is pumped to the pond. Horizontal inlets should be used for gravity flow. When upward discharging lines are used, the discharge end of the pipe should be located approximately 1 foot above the bottom of the pond and should not extend to such elevation that ice will damage the terminal structure during winter operation. The end of the discharge line should rest on a suitable concrete apron with a minimum size of two feet square. Manholes or cleanouts are recommended where the inlet pipe passes through the embankment.

Influent lines should be placed on or under the bottom. The use of exposed dykes carrying influent lines to the center of the pond should be prohibited, as such structures will impede circulation.

Overflow Structure:

The location of the outlet near the windward shore should prevent any wind induced short circuiting and permit maximum time-distance between inlet and outlet. Maximum flexibility in stabilization pond operation may be facilitated by incorporating pond level and effluent draw-off level controls in the overflow. An example of such controls is a stop plank support in a manhole for pond level control and an adjustable level intake structure to the manhole.

Multiple Ponds:

The use of multiple cells to provide greater flexibility of operation is desirable. Multiple cells permit both series and parallel operation which offers flexibility in handling fluctuating loads such as are experienced in communities that have a large seasonal influx of tourists. It has also been found that erosion due to wind action has been much less on small two-cell installations than at large single pond construction.

Fencing and Signs:

The installation of fencing and "No Trespassing" signs is mandatory. The purpose of the signs is to notify persons of the nature of the facility and discourage trespassing. The minimum requirement for fencing is that it be stock-tight and at least 6 feet high to keep out animals and unauthorized persons.

Pretreatment:

Usually, treatment of the raw sewage prior to application to the stabilization pond is omitted. Although some savings in land costs may be realized by reducing the B.O.D. loading to the pond by some means of preliminary treatment, the initial cost of a primary sedimentation unit plus the operating and maintenance costs usually offset any saving.

Normally, chlorination of the pond effluent is not required. However, chlorination treatment is sometimes provided where protection of the receiving waters is necessary during the recreational months.

AERATED LAGOONS

Since the necessary areas may not always be available for the conventional type lagoons, especially for high B.O.D. loads and flows, it may not always be possible to take advantage of the lagoon type of treatment. As a result, consideration has now been given to the supplying of varying proportions of oxygen to such lagoons by mechanical means.

The use of turbine type mechanical aerators, which disperse and mix compressed air into the liquid mass, permits oxygen absorption efficiencies of a much higher order. The major advantage of the aerated lagoon is the continuous oxygen transfer caused by the turbine aerator. In normal oxidation ponds, the algae produce oxygen only in the daylight hours so that sufficient oxygen must be stored during the daylight hours to satisfy the micro-organisms through the night. The continuous oxygen supply of the mechanical aerator, 24 hours a day, permits the aerated lagoon to handle more waste water per day per unit volume.

The surface aerators can be installed on fixed platforms or on floating rafts which are properly anchored. The agitation provided will prevent freezing in cold climates and also keep the pond aerobic even if a portion of the pond is ice covered. The aerator developed by the Infilco Company, known as the "Vortair" aerator, consists of a specially designed turbine located adjacent to the water surface. It pumps up liquid from the bottom of the lagoon and discharges it radially at the surface, creating a so-called peripheral hydraulic jump entraining large quantities of air. Variable speed can be provided to adjust the oxygen input to the particular load.

By the use of artificial aerated lagoons, treatment can be practised with only a four to six day retention time. With solids return from a clarifier following the aerated lagoon, total oxidation type treatment could be practised with a one-day retention period. It is also possible to utilize an aerated lagoon in conjunction with a conventional oxidation pond to obtain the benefits of both systems.

Like all waste treatment systems, the aerated lagoon has certain advantages and disadvantages. It appears to have some value where oxidation ponds are overloaded or where property is expensive. The only maintenance required is periodic lubrication of the aerator motor. There are certain problems to be solved in the use of aerated lagoons for sewage treatment but as installations are constructed, more data will become available and design criteria will become formulated. Currently, we are considering two such proposals in Ontario, one for the treatment of potato wastes where conventional lagoons were unsuccessful due to excessive loadings and the second installation for the treatment of sanitary and industrial wastes from a small municipality where soil characteristics are unsuitable for conventional lagooning.

PUBLIC HEALTH CONSIDERATIONS

The same precautions which are used in the operation of conventional sewage treatment plants should be practised with waste stabilization ponds. Even though the reduction in bacteria in lagoons is quite high, the possibility of an infection by contact with the sewage should be recognized. Thus, the need for adequate fencing to prevent access by children and animals as well as the posting of signs prohibiting trespassing.

Ground water supplies, particularly those used for municipal purposes, should not be accessible to the seepage from stabilization ponds. Questions have also been raised regarding the possibility of infection being transported by wild fowl which frequent these ponds as well as the infection of livestock by watering in streams receiving lagoon effluent. As yet, no significant data have been established to substantiate these possibilities.

Surveys of mosquito breeding in lagoons in the Dakotas have concluded that production will be of little consequence if weed growths are prevented or eliminated and larvicide is used as required, particularly, if difficulty occurs during initial filling.

ECONOMICS OF WASTE STABILIZATION PONDS

The feasibility of waste stabilization ponds depends largely upon the availability of suitable land. Initially, the development of lagoons may have been retarded somewhat because of the belief that land costs could exceed other financial benefits, such as low operation and maintenance costs and the initial capital cost when compared with other treatment methods. Experience has shown that in many cases the price of land may be about 50 per cent of the cost of the completed waste stabilization pond, yet the total cost has been equal to or less than the cost of a completed secondary treatment plant. In numerous instances, land costs could be double or triple the completed lagoon construction costs before equaling the conventional plant cost.

Experience has also shown that in additon to providing advantages of a high degree of treatment, low initial capital cost and low operation and maintenance cost, the waste stabilization pond is quite flexible in areas which are subject to rapid population growth. It has been found that lagoons may be re-sited and constructed downstream and the pond area which has appreciated in value may then be reclaimed for housing or industrial site development.

In addition to being an economical method of sewage treatment, the waste stabilization pond has been useful in providing a polishing of conventional, primary or secondary treatment plant effluents. There are two municipalities in Ontario in which it has been found to be more economical to abandon existing conventional treatment works and construct waste stabilization ponds to handle the entire sewage flow from the community. In both these instances, it was not possible to obtain an adequate area for the lagoon at the plant site, but suitable land was found downstream at a reasonable cost.

The data in the following table may be used to provide a cost comparison of the approximate cost of each type of sewage treatment facility. It is apparent that there is a need for compiling more complete project data which will assist in evaluating preliminary engineering phases.

Treatment Plant Costs Based Upon Population Equivalent

Population Equivalent	Primary	Secondary	Lagoons With Interceptor	Lagoons Without Interceptor
100	$77.91	$127.30	$69.90	$29.36
1,000	$40.05	$ 64.83	$34.79	$18.52
10,000	$20.58	$ 33.02	$17.32	$11.69

All costs include land.

Adequate records of operation and maintenance costs have not been gathered for waste stabilization ponds. Lagoon maintenance seldom requires more than two cuttings of the grass on the dyke walls per year. The removal of scum, grease and floating material is generally no problem. Therefore, the operation does not require the attention associated with the removal of the same material in conventional treatment plants. Seasonal operation of valves may be required for multiple cell installations or where variable depth control is exercised. Removal of emergant vegetation may be required periodically but this may be minimized by providing the required liquid depth. The estimated operation and maintenance costs for waste stabilization ponds ranges from 20 cents to one dollar annually per population equivalent, and from one dollar to four dollars per population equivalent per year for the conventional type of plants.

These factors do not mean that costs are always less for lagoons. Many details such as longer outfall sewers, high pumping costs, high original land costs or difficult construction features may raise costs well above those for the conventional plant. However, a feasibility cost study will indicate the economic advantages of waste stabilization ponds for each installation.

CONCLUSIONS

In reviewing the accomplishments of waste stabilization ponds throughout the country, the following conclusions are apparent:

1. Research and field investigations have definitely proved that waste stabilization ponds are a practical and economical method of sewage treatment.
2. Properly designed and operated stabilization ponds may be expected to provide a degree of treatment comparable to conventional sewage treatment processes.
3. Although stabilization ponds appear to be simple in design and operation, it is essential that the climatic and geographic factors be considered in establishing design criteria.
4. On the basis of operating experience in the western provinces and the United States, higher organic loadings are feasible where ice-free climates prevail or where odour conditions are not a controlling factor. Until such time as operating data are available in Ontario, a loading of 20 lbs. of B.O.D. per acre should be used.
5. The factors to be considered in site selection for waste stabilization ponds are essentially the same as conventional types of sewage treatment plants.
6. Although there is no evidence to indicate that stabilization ponds constitute a public health hazard, it appears that there are greater potentialities in the propagation of insects and other possible disease vectors than at conventional treatment works.

ONTARIO WATER RESOURCES COMMISSION

INTERMEDIATE SEWAGE WORKS OPERATORS' COURSE

March 9, 1962

Name____________________________ Municipality____________________

1. A pump lowers the liquid level in a tank 1 foot in 30 minutes. This tank is 40 feet long by 20 feet wide. What is the capacity of the pump in cubic feet per minute?

2. Under the headings provided, list two effects of the discharge of improperly treated sewage on the receiving stream.

(a) Effect on Chemical condition
(1)
(2)

(b) Effect on bacteriological condition
(1)
(2)

(c) Effect on biological condition
(1)
(2)

3. (a) What are the average values of the following in a normal domestic sewage?

B.O.D. ______________________ p.p.m.

Suspended Solids ______________________ p.p.m.

(b) List two nutrients which are of importance in the activated sludge process:

(1)
(2)

4. (a) List three methods of disposing of screenings:

(1)
(2)
(3)

4. (b) Name <u>three</u> types of grit removal facilities:

(1)
(2)
(3)

5. (a) What is the most significant analytical analysis in sewage plant control?

(b) What pH should be maintained to give the optimum digester performance?

(c) What minimum velocity should be maintained in sewers to prevent the deposition of solids?

6. (a) Draw a sketch to show the step aeration process.

(b) The volume of mixed sludge to be removed from the primary settling tanks increases as the suspended solids concentration of the digester supernatant increases -

True __________ False __________

(c) The greater the concentration of volatile solids in the mixed liquor, the greater will be the poundage of volatiles to be wasted in the activated sludge process -

True __________ False __________

(d) List <u>two</u> possible causes of sludge bulking and indicate the correct action which should be taken to overcome this condition:

7. List <u>two</u> reasons for the lack of discharge from a centrifugal pump and indicate the corrective measures for each which should be taken to restore the pump to normal operation:

8. (a) Give <u>three</u> examples of tertiary treatment:

(b) What loading factor is used in Ontario for determining the area requirement for a waste stabilization pond:

9. Check the most correct statement:

(a) In a single stage digester operation the supernatant should be withdrawn

(i) when quiescent conditions exist.
(ii) when the temperature of the digester is between 90 - 95°F
(iii) when the mixers are being operated.
(iv) when the volatile acids level is below 2000 p.p.m.

(b) Check the two items which should receive the most consideration in operating a tank truck sludge disposal program:

(i) public relations.
(ii) price received for sludge.
(iii) distance from homes and possible odour nuisances.
(iv) sludge pH.

(c) Check the amount of gas production which should be expected from a primary plant serving 10,000 persons.

(i) 1,000 cu. ft. - 1 day.
(ii) 10,000 cu. ft. - 1 day.
(iii) 100,000 cu. ft. - 1 day.

10. (a) Give four rules that should be followed in maintaining electrical apparatus.

(b) Why is infiltration in sewers receiving more attention?

(c) What difficulties does infiltration cause at sewage treatment plants.